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Cement and Concrete Control
San Francisco-Oakland Bay Bridge*

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GENERAL PROBLEMS

The bulk of the concrete in the San Francisco-Oakland Bay Bridge is exposed to the direct or indirect action of sea water. Some 500,000 cu. yd. are involved in the piers with bases founded on the bottom of the Bay under varying depths of water and mud to a maximum of 240 feet below water surface, and, in the case of the central anchorage—Pier 4, towering more than 240 feet above the water.

The concrete in the remainder of the project,—more than 200,000 cu. yd., is either adjacent or close to the shores of the Bay and across Yerba Buena Island, all subject at least indirectly to the action of sea air or water.

Great care was taken in the preparation of the specifications for this project and in the subsequent fabrication of the concrete, to the end that the highest quality product with greatest practicable density might be secured with a cement content for the most part of five sacks per cu. yd. This care has extended not only to the design and fabrication of the concrete mix but likewise to the specifications for the cement.

CEMENT

While it is true that there are a number of marine structures adjacent to the bay and ocean shore in the San Francisco region, which, even though constructed of cement relatively high in the compound tri-calcium aluminate, have withstood the destructive action of sea water for periods of thirty years and more without showing appreciable deterioration; nevertheless, because of the early deterioration of concrete in many marine structures, and of the recent studies by Lerch and Bogue, and by other well-known investigators of the action

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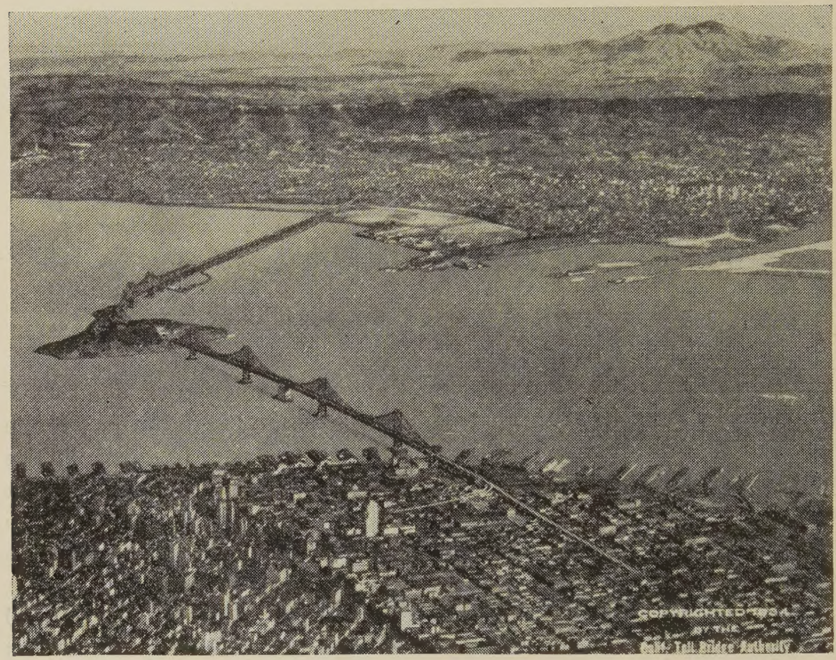


FIG. 1—AS SAN FRANCISCO BAY WILL LOOK IN 1937

Upon an aerial photograph of San Francisco Bay, with Oakland in the background, architects for the San Francisco-Oakland Bay Bridge have drawn to scale a representation of the world's largest bridge, 8¼ miles long, (nearly four miles over water) which will connect Alameda and San Francisco counties.

The west half of the bridge is a suspension structure comprising twin suspension bridges anchored into a huge concrete monument in the center.

A double-deck tunnel pierces Yerba Buena Island, occupied by Army, Navy, and Lighthouse services, and the double-deck bridge continues over a 1400-ft. cantilever span, 5 through truss spans, and 14 deck truss spans before it lands on a fill extending out from the Oakland shore.

At the eastern shore, trestles carry the bridge traffic on to three branches—one for Berkeley, one for Oakland, and one for the business section of Oakland and Alameda.

The piers of this bridge—51 in number—set new marks on engineering frontiers, going deeper below water than any previous substructure has heretofore been built. Some of the piers go as far as 237 feet below low tide.

The two suspension bridges have 2310-foot main spans.

The lower deck carries two tracks for interurban electric cars and three lanes for heavy trucks, and the upper deck carries a 58-foot highway for six lanes of automobiles.

The bridge is being built for the California Toll Bridge Authority, of which Governor Frank F. Merriam is Chairman, by the State Department of Public Works under Earl Lee Kelly, Director. C. H. Purcell is Chief Engineer.

of sulphates on high C_3A portland cement, the Department favored a cement relatively low in this particular compound.

However, because there was some uncertainty regarding the ability of the local cement companies to furnish this special type of cement within the requisite time limit, the first contracts specified standard portland cement without limitation.

Before work was actually under way, it developed that several companies were in a position immediately to furnish low C_3A cement at no increase in price over standard. Therefore, before the first concrete was placed the specifications were amended to read as follows:

> Description—The cement shall conform to the requirements of the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials, Serial Designation C9-30, with the following exceptions and additions:
>
> (a) The percentage of tricalcium aluminate shall not exceed eight per cent (8%) by weight, when computed from the cement analysis by the method of Bogue, as published in the Analytical Edition, *Industrial and Engineering Chemistry*, Vol. 1, No. 4, P. 192, Oct. 15, 1929.

The specifications were further modified to permit the use of an approved high silica cement with the requirement that there should not be more than 8 per cent C_3A in the clinker.

To meet this specification the relatively high alumina and low iron content of the raw materials at three of the plants manufacturing cement for the Bay bridge require the addition of iron in the form of iron scale or iron ore, as well as the addition of increased amounts of silica (depending on the chemical analysis of the raw material).

The four companies furnishing cement for the bridge have met the low C_3A requirement, therefore, other than in the case of the first few carloads of cement, which averaged a little over 9 per cent all of the portland cement clinker used in the manufacture of Bay bridge cement has averaged substantially less than 8 per cent tricalcium aluminate, running, in some cases, as low as 2 per cent and averaging 5 per cent.

The Santa Cruz Portland Cement Co. of California started the manufacture of a "High Silica" hydraulic cement for commercial use prior to 1932.

A small quantity of this cement was used in the construction of some concrete box culverts in Mariposa County, California, during June and July, 1932.

In September of the same year, 1000 lin. ft. of 20-ft. width, hydraulic concrete pavement was constructed on the Bayshore Highway South of San Francisco, using Santa Cruz high silica cement. The usual laboratory tests for soundness and strength indicated this cement to

Fig. 2—Operations on San Francisco-Oakland Bay Bridge

Aerial view of traveler derrick erecting Span E-9 (lower left) which is the last of the 288-foot spans. Also, erection of Span YB-1 on Yerba Buena Island in progress. The entire West Bay Crossing, with Tower W-5 under construction may be seen in the background.

be the equal of standard portland cement and there was some reason to believe that owing to the pozzolanic nature of the added silica compound it might be a more durable product in the presence of sea water than any of the standard commercial portland cements manufactured by California mills.

In the manufacture of Santa Cruz high silica cement, portland cement clinker is made in the usual manner after which a silicious compound, made by calcining a mixture of silicious material and lime, is ground with the clinker; the silica ranging from 80 to 90 per cent of the total compound.

Two grades of high silica cement have been used in Bay bridge concrete; one grade containing 30 per cent, another 15 per cent of silica-lime compound.

94 lb. of 70-30 H. S. cement measures 1.24 c.f. Sp. Gr. 2.90
94 lb. of 85-15 H. S. cement measures 1.12 c.f. Sp. Gr. 3.07

Low C_3A cement conforming with A. S. T. M. standards has been, or is being used in the construction of all of the main piers; the tunnel, anchorage and approaches on Yerba Buena Island; and the roadway decks on the steel superstructures involving more than 500,000 cu. yd. of concrete. High silica cement is being used in the construction of the San Francisco anchorage and approaches, and also the approaches at the Oakland end of the bridge, involving more than 125,000 cu. yd.

Following is the average composition of the special low C_3A and high silica cements furnished by the four companies supplying cement to this project.

	Calaveras	Pacific	Yosemite	Santa Cruz		
				30% Comp. H. S.	15% Comp. H. S.	Clinker
CaO	63.00	64.35	65.80	53.28	57.12	64.99
MgO	2.92	2.09	2.23	1.91	2.06	2.18
Al_2O_3	4.90	5.21	4.26	6.62	6.04	5.84
Fe_2O_3	5.22	4.94	2.08	4.46	4.78	5.27
SiO_2	21.47	21.18	23.53	32.01	28.01	21.04
SO_3	1.21	1.49	1.16	1.29	1.51	0.11
Loss	1.58	0.90	0.93	1.35	1.29	0.85
Insoluble	0.16	0.13	0.23	15.82	9.99	0.28

Tests of cylinders up to one year and of pavement cores up to 2 years, from concrete of the Bayshore Highway containing 6 sacks per yard, have shown these low C_3A and high silica cements to be practically of the same strength as the regular cements manufactured by these companies. Fruthermore, these cements have shown high resistance in the Merriman Sulphate Test, whereas the regular cements having more than 8 per cent C_3A failed to pass this test. The same special cements show decided superiorities to the regular cements in a special test series now under way in which concrete specimens have been partially immersed in a strongly alkaline soil from the Sacramento Valley.

CONCRETE

The problem of fabricating a uniform high quality concrete was somewhat greater than that encountered on the usual construction project, because of variations in the source of aggregate supply and methods of batching and placing.

The problems can be classified under the following general headings: (1) Aggregates; (2) Batching; (3) Transportation; (4) Placement.

Aggregates

The maximum size of coarse aggregate fixed for this project was 3 in., even though a larger maximum could have been used to advant-

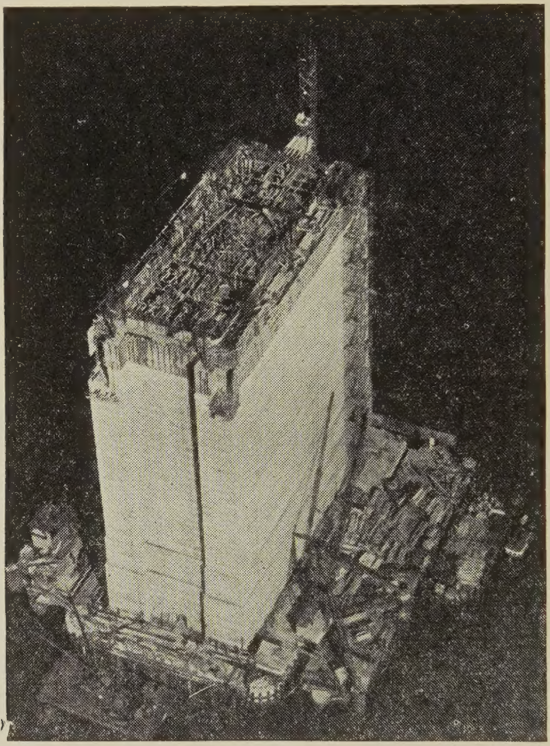


FIG. 3—PIER W-4 PLACED AT ELEVATION OF 222 FEET

Pier W-4, concrete center anchorage of the twin suspensions spans (West Bay crossing) showing concrete placed to an elevation of 222 ft. above water level and forms for lower deck slab girders being placed. The height of the concrete center anchorage from water level to top of concrete will be 235.18 ft.

age in some of the mass concrete. The main reason for this limitation was the deficiency of gravel exceeding 3 in. in and around the San Francisco Bay region.

Although there are several aggregate plants operating in ledge rock and, therefore, in a position to furnish any maximum size desired, it was felt that there was not sufficient advantage in specifying a larger maximum size to justify a specification which would force the use of crushed ledge rock and discriminate against more than 90 per cent of the aggregate producers in the field.

Grading limits were set up for two sizes of coarse aggregate, designated as "2½ in. to 1½ in.", and "1½ in. to No. 3." In beams, slabs, rails and all sections less than 8 in. thick or having reinforcing bars less than 4 in. apart, and within the structural steel cutting edge

sections of the caissons, the maximum aggregate size was 1½ in. In all other parts of the structure the maximum size was 2 ½ in.

Grading of the fine aggregate was also specified within tolerances, but the contractor was permitted to furnish this in two sizes, designated as pea gravel and sand, respectively, which when blended either at the plant or the job, would produce the grading specified.

SPECIFICATIONS

The Combined Aggregate for concrete shall be so graded that the percentage composition by weight, as determined by laboratory screens and sieves, will be as shown in the following table:

COMBINED AGGREGATE GRADING

Passing A	2½" Maximum Per Cent	1½" Maximum Per Cent
3" circular opening	100	———
2½" circular opening	95 to 100	———
2" circular opening	83 to 90	100
1½" circular opening	72 to 83	95 to 100
¾" circular opening	56 to 70	65 to 75
3 mesh sieve	30 to 40	33 to 43
10 mesh sieve	18 to 27	20 to 30
20 mesh sieve	10 to 18	11 to 20
30 mesh sieve	6 to 12	6 to 12
40 mesh sieve	3 to 8	3 to 8
80 mesh sieve	0 to 4	0 to 4
200 mesh sieve	0 to 2	0 to 2

The combined grading of fine and coarse aggregates shall control the grading of any separate groups of sizes proposed for use. The size composition of the combined mixture shall show a uniform increase from coarse to fine materials as measured by the percentage retained upon a set of standard laboratory screens and any individual groups of sizes which do not lend themselves to this size composition of combined mixtures shall be considered not to have met these requirements.

Variations in moisture content in any group of sizes of aggregate shall not exceed one per cent (1%) of the weight of the aggregates in a saturated surface dry condition. Variations in specific gravity of any group of sizes of aggregate shall not exceed one per cent (1%). Variations in grading of the separate groups of sizes of the various aggregates shall not exceed five per cent (5%).

Variations exceeding the maximums as stated above shall constitute cause for delaying the use of the materials until batch weights and mixing water can be adjusted for the variations.

The specifications further classified the concrete according to strength, density and maximum water content, workability and uniformity—the classes of concrete being designated as follows:

CLASSES OF CONCRETE

Class	28-Day Strength	Bags of Cement Per Cubic Yard	Water Max. Cem. Ratio By Volume
A-A	4000	7	0.65
A	3200	6	0.75
B	2500	5	0.85

Prior to the preparation of the specifications, a survey was made of the available supply of aggregates for the project. Wasting and rehandling, it was pointed out by producers, would result from the sorting out as usually required on state projects. Pit run materials varied widely in size composition, with a dearth of the larger sizes required by theoretical economy in use of cement. It was conservatively estimated that for every one hundred cars of pit run material put through the processing plant, only fifty could be used for concrete under conditions of design and control in use previous to the preparation of concrete specifications for the Bay bridge.

To avoid the higher prices and the production problems that would develop from this condition, specifications were made to fit a greater range of materials but with added emphasis on the control requirements. The aggregate production facilities were pooled and operated under the control of a central agency. With some changes in specifications for concrete and concrete practice to permit shipments from several sources, the state gained several advantages from this pooling of production of aggregates.

The Department proposed a method for estimating the quantities of each grade of material required for concrete of known strength, density, and weight per cubic foot, which method permitted the use of available materials. This method of design and control provided the limits to the use of pit run materials and is an effective remedy for a condition that tends to place a premium on selected aggregates.

A specification for concrete containing a specified cement content will provide for definite strength and density if the water is definitely limited. Any combination of sizes which can be fabricated with this cement and water content will be of equal strength and density. The only effect of changes in size composition is a change in workability. If ample allowance of cement is made to conform to the use of an ample portion of mortar, the grading of the coarse aggregate may vary without any appreciable effect upon workability. The control of these quantities can be assured if batching is by weight and proper consideration is given to the specific gravity and moisture content. These details were clearly defined and provided for in the specifications.

These methods and factors had been thoroughly tested by the state and the economic advantages confirmed by producers prior to the award of contracts for the Bay bridge project. The producers were, therefore, prepared to lend their support to the principles of cooperative production and delivery, and, with faith in the administration, prices were established accordingly.

The Concrete Products Sales Co. was organized to buy, distribute and sell concrete and concrete materials. This company submitted a proposal to contractors on the Bay bridge to deliver mixed concrete to the various units of construction. This proposal was accepted for all construction involving concrete delivery by water transportation.

For the reasons given above, the specifications permitted the use of materials having a wide range of characteristics. However, modifications take time and delay operations or cause expanded plant investments. To avoid or minimize delays, state inspectors were placed at all aggregate-producing plants from which materials were obtained.

Shipments of aggregate were made from the following plants:

1. P. C. A. Plant "C," Elliot, Calif.
2. P. C. A. Plant "D," Elliot
3. P. C. A. Plant "E", Niles
4. Kaiser Paving Co., Radum
5. Blake Brothers, Richmond
6. Basalt Rock Co., Healdsburg
7. Kaiser Paving Co., Antioch
8. California Rock Co., Elliot

The inspectors tested all shipments for identification and classification as related to batching plant operations essential to a satisfactory combined mixture. State inspectors working with the plant superintendents succeeded in obtaining similar products, from each of these plants, thereby causing a minimum of delay at the batching plant. This was done by adjusting the water flow through the sand classifiers and changing screen openings to fit a specific grading for each group of material.

Each producing plant had a card color to define similar grading from different sources. The train crews then blocked all cars with similar cards into groups in the train. These blocks of cars were held together during shipping and switching, and were spotted at the batching plant from switching lists made from manifest sheets forwarded from the aggregate plant inspector to the batching plant inspector. This arrangement for handling cars of similar materials minimized the number of changes in proportion, and removed some of the hazard of delay in batching plant operations.

BATCHING

Handling Aggregates at Batching Plant

The component parts of the concrete were weighed out at batching plants placed at convenient positions near the work. There were five of these plants, the highest hourly demand being upon the Kaiser Marine Plant No. 1.

This plant was located in the Southern Pacific Asiatic Pier No. 2 on the Oakland side of the Bay. It was served by four stub railway tracks. Three of these tracks were used to bring in aggregate and one

FIG. 4-7
SAN FRANCISCO-OAKLAND BAY BRIDGE

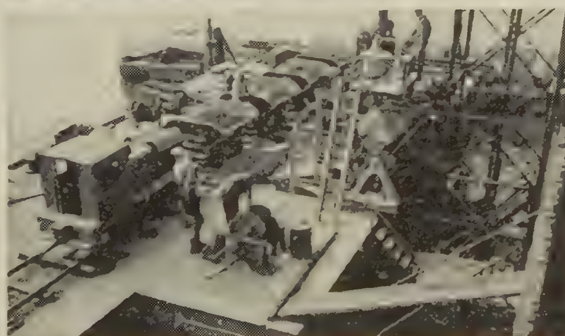


Fig. 4 (upper left) Mixing and receiving barges. Fig. 5 (upper right) Agitator, pumps and pipe line for pump concrete caisson, Pier W-6. Fig. 6 (lower left) Method of handling aggregates for paving roadway decks—Materials batched into cars at batching plant, hauled to mixer at base of steel superstructure; mixed concrete elevated by skip and tower to upper deck and where dumped into receiving hopper and then into batch cars for transportation to place of deposit. Fig. 7 (lower right) Tower for handling concrete, west anchor pier cantilever span, East Bay crossing.

to bring in cement. Barges of crushed rock and cement were tied alongside. At the end of the pier and beyond the end of the stub tracks was the batching plant.

The limited amount of coarse aggregate at the gravel plants necessitated shipments of crushed rock from quarries. This crushed rock was hauled by water on flat barges. A two and one-half yard clam shell bucket and crane recovered from the barges.

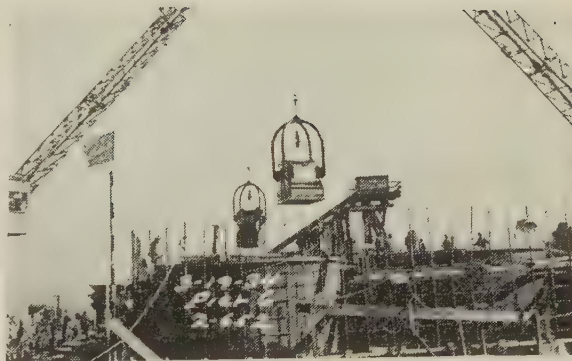
Numerous difficulties were encountered with harsh mixes due to stone chips. The demand for concrete and the shortage of coarse gravel frequently forced the inspectors to use materials which were objectionable from the standpoint of workability. As quickly as better equipment could be delivered, the crushed rock producers installed additional screening facilities between the quarry storage and loading terminal. These screens were operated to remove over and under sizes from each group. A shuttle type conveyor was installed at the barge loading terminal, which minimized fragmentation due to falling of the materials, and segregation was eliminated by operating the shuttle and the barge back and forth in such a manner that the rock was placed on the barges in layers.

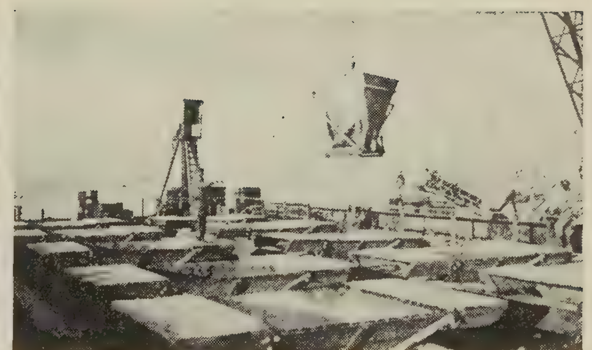
The card system for marking car shipments was in operation but a short time when it became evident that aggregate-processing equipments and methods were not accurately controlled, and shipments varied over the entire range of the grading limits. This condition required long hours of testing to identify the groups, numerous switching moves to gather cars of similar identification, and delays at the batching plant for mix adjustment.

The superintendent of the Concrete Products Sales Co. called all producers of aggregates to a conference at the field laboratory office. After the operators had stated their problem of making uniform materials, group gradings which would meet with state approval were selected as standards of production for all plants. The representatives of the producers agreed to make such changes in equipment and methods as were necessary to produce these groups. Within a few weeks, and with the cooperation of the state inspectors, the correct operations and screen sizes were fixed for each plant, thereby giving a product from all plants that could be used within the limits of less than five different proportions for any type of workability.

Delays in batching due to variation of materials became the exception in routine batching operations. The effectiveness of these processing methods made it possible eventually for the state to remove inspectors from the plants to other work, thereby lowering inspection

FIG. 8-11—
S. F. O. B.
BRIDGE





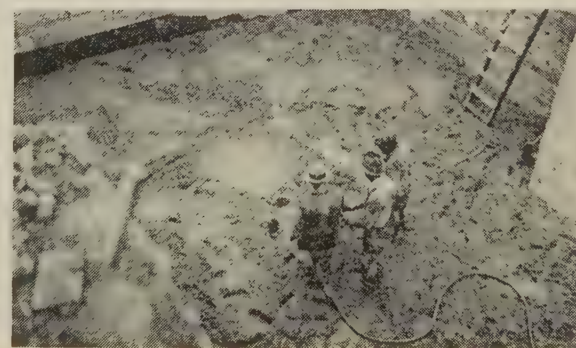


Fig. 8 (upper left) Placing seal concrete. Pier W. 6. Pipe guides over the top of the bucket prevent fouling under the cross walls of the caisson. Fig. 9 (upper right) Placing concrete in caissons, East Bay crossing.

Fig. 10 (lower left) Showing method of handling concrete, East approach structure. Fig. 11 (lower right) Vibrating concrete San Francisco anchorage.

costs. Thereafter an adequate supply of aggregates for faster batching operations was assured, and other problems of producing more concrete were given greater attention.

General Batching Plant Design

The general requirements of the specifications to be considered in designing and detailing the batching plants were that they be of ample capacity, that equipments be ruggedly constructed and that equipment and essential operations be interlocked against duplications or quantitative errors. All equipments selected for these installations were standard types which had proved practical qualities and characteristics. The plant operations were successfully interlocked in different ways at the several plants erected under these specifications.

The general plan of all plants followed the standard system of providing bunkers over the weigh hoppers for four groups of sizes of aggregates and silos for cement storage over the cement-weighing equipment. All plants followed standard design in this respect and the problems are familiar ones.

Batching Equipment

Before drawing the specification for the Bay bridge, a thorough study was made of the success had with automatic weigh batching equipment installed on various major projects including the Boulder Dam and it was decided that this type of equipment had reached the stage where its operation was dependable and the advantages to be gained by the insurance of absolute uniformity of control of the concrete mixes justified its mandatory use. The specifications, therefore, provided that the amount of cement and the fine and coarse aggregate should be determined by direct weighing equipment which should include a visible dial or equally suitable device accurately registering the scale load of any ingredient at any state of the weighing process; that the dials should be operated by a mechanism of the full automatic type capable by means of a photo-electric cell or other suitable device of actuating the gates controlling the flow of the various ingredients into the weigh-hopper at previously determined weights so as to provide accurate proportioning to within one per cent of the specified weight of cement and two per cent of the specified weight of each separate aggregate.

It was provided that the weights of the various ingredients should not be cumulative on the dial but that the dial should be graduated to accommodate the heaviest ingredient and a poise provided for each ingredient in such a manner that when the poise is set on the beam to the required weight the indicator on the dial shall point to that weight and as the material flows into the weigh-hopper, the indicator shall

FIG. 12-15—
S. F. O. B.
BRIDGE



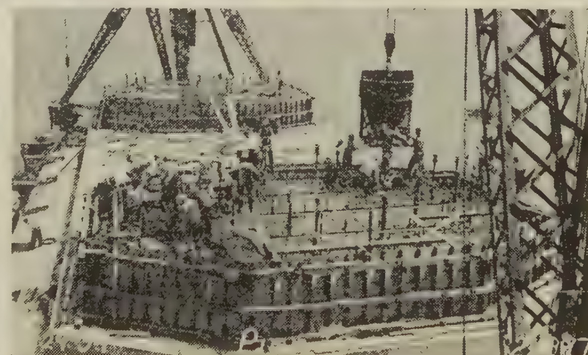



Fig. 12 (upper left) Type of bucket used for placing concrete in anchorage. Fig. 13 (upper right) Buckets used for placing concrete in the dry East Bay piers.

Fig. 14 (lower left) One type of bucket used for depositing concrete in the dry. (Fig. 15 (lower right) Placing concrete in pedestal base.

approach "0", reaching "0" when exactly the right weight has been attained. In this manner the weight of each ingredient returns the indicator to "0," cutting off at that point.

It was further provided that the equipment should include an accurate automatic graphic recorder capable of being locked, for visibly and graphically recording the time of weighing and the actual weight of each separate ingredient, including water entering into every batch as well as the weight of surface moisture.

The equipment should be capable of ready adjustment for determining the varying weight of surface moisture in the sand and coarse aggregates in every batch and of compensating in each batch for the varying weight of moisture contained in the aggregate. It should also be capable of ready adjustment for changing the proportions of batch satisfactorily.

The equipment was arranged so as to permit the convenient removal of over-weight material in excess of the prescribed tolerances.

No difficulty was encountered in securing equipment which would conform with the specifications outlined above and it has satisfactorily performed throughout the life of the project in four batching plants where materials for more than 500,000 cu. yd. of concrete have been accurately and uniformly batched with negligible delays due to faulty equipment operation or breakdown.

TRANSPORTATION

For all land concrete, batching plants were set up close to the site of the work with a concrete mixer as a part of the batching plant construction. Mixed concrete was thereafter transferred from the mixer into the construction unit either by dumping from the mixer into a bottom dump bucket swung into place by a crane or by dumping into a truck or car where necessary to transfer the concrete to a unit of the structure some distance from the plant.

For all water structures, however, involving the bulk of the concrete, a different method of transportation was required. The final plan adopted by the sub-contractors for the concrete batching and transportation was to batch all materials at a central batching plant in batches containing aggregate and cement sufficient for 3½ cu. yd. of concrete. Single batch bins were erected on barges for combined rock and sand and companion metal tanks to hold the required amount of cement. The maximum capacity of any barge was 80 3½-yd. batches, or 280 cu. yd. of concrete.

Each bin was equipped with an opening controlled by a gate at the bottom, through which the material was dumped onto a belt which conveyed the aggregate and cement to mixers at one end of

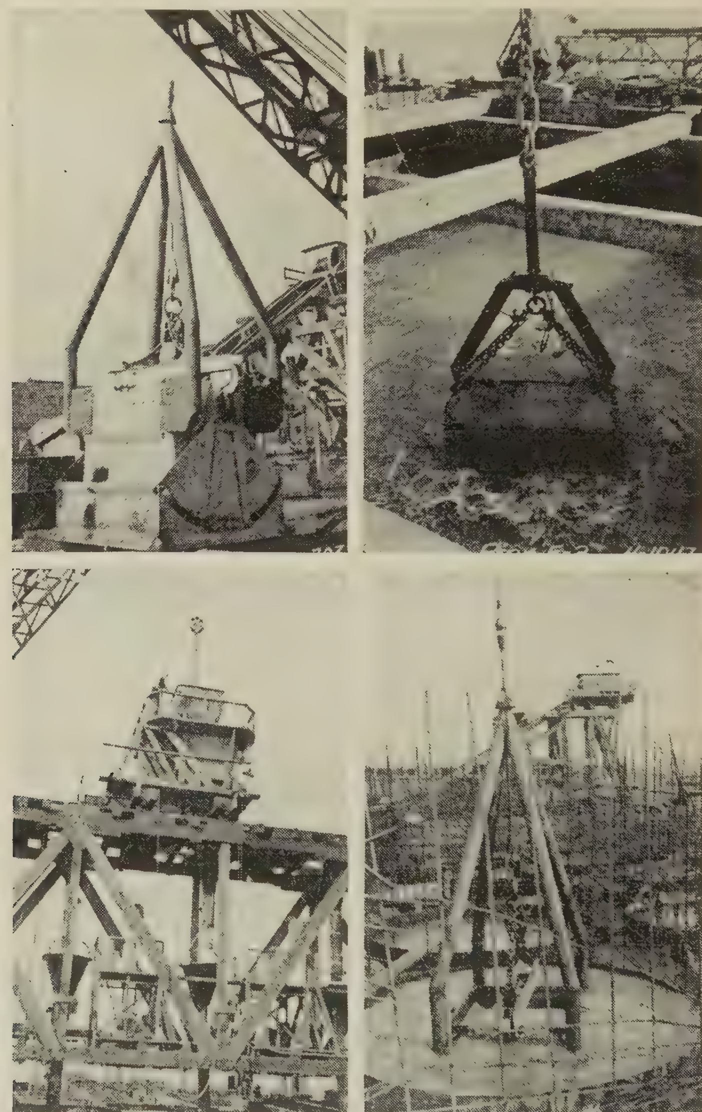


FIG. 16-19—CONSTRUCTION OPERATIONS S. F. O. B. BRIDGE

Fig. 16 (upper left) Bucket for depositing under water seal concrete, West Bay piers. Fig. 17 (upper right) Type of bucket used for placing under water concrete at depth of 230 ft., East bay piers. Fig. 18 (lower left) Arrangement for placing tremie concrete, East Bay piers. Fig. 19 (lower right) Lowering bucket filled with concrete for seal Pier W-6. Pipe guides over the top of the bucket prevent fouling under the cross walls of the caisson.

TABLE 1—SUMMARY OF 28-DAY CONCRETE TESTS—ALL UNITS. SAN FRANCISCO—OAKLAND BAY BRIDGE TO JUNE 1, 1935

Total to Date								
	CLASS A (6 Sk. Cement)				Class B (5 Sk. Cement)			
Cont.	No. Spec.	Struct. Comp. Strength	No. Spec.	Under Water Comp. Strength	No. Spec.	5-Sk. Struct. Comp. Strength	No. Spec.	5½-Sk. Struct. Comp. Strength
Sea Water Cement								
2	59	4051	237	3894	704	3515		
4	215	4104	260	3565	181	3372		
5	126	4392			85	3711		
Ave.		4158	497	3722	970	3506		
High Silica Cement								
3	16	4099			111	3511		
8	3	4263	49	4089	19	3796	68	3929
Ave.	19	4125	49	4089	130	3553	68	3929

TABLE 2—TYPICAL RELATION OF BATCH VOLUME—MEASURED VOLUME AND CEMENT YIELD (6-SACK CONCRETE) SAN FRANCISCO—OAKLAND BAY BRIDGE
Illustrating accuracy of control under automatic weight batching system
UNIT—Pier No.

	18	19	20	21	22
Batch Volume (cu. yd.) (From Batch Weights)	4855.39	4760.36	4341.25	4466.04	4257.28
Measured Volume (cu. yd.) (Concrete in Place)	4849.00	4724.83	4314.05	4423.19	4239.74
Sks. of Cement (Total)	28,911	28,431	36,025	25,505	25,389
Sks. Cement per cu. yd. Concrete	5.962	6.018	6.035	5.992	5.988

the barge. There were four of these mixers on each of the larger barges, more than twice the required capacity to avoid delays due to mixer breakdown. After mixing, the concrete was conveyed by belt to the opposite end of the barge where it was dumped onto an inclined belt mounted on a unit which could be swung around over the work platform at the construction unit where the concrete was dumped from the barge conveyor belt either into a hopper and thence into a bucket for crane placement into the work, or onto another traveling belt which conveyed the concrete to a tower where it could be dumped into a hopper and subsequently distributed to the work as desired.

There were many operating problems to be worked out with this type of equipment which were a source of grief to the engineer as well as the contractor. Once a barge containing 80 batches of materials

left the batching plant no changes in proportioning in successive batches could be made after arrival on the work to meet changing conditions both in the materials and method of placement.

The troubles inherent in belt transportation of concrete were even more acute on this, than on the usual project where belt transportation is used.

Through experience, these problems were gradually reduced to a minimum, however, and the general results were very satisfactory.

Table 2 gives some idea of the accuracy of control developed.

PLACEMENT

It was necessary to design suitable 6-sack concrete mixes for underwater concrete placed by Tremie to depths of 50 ft. and by bottom dump bucket to depths of more than 240 ft., also 5- and 6-sack concrete placed in the dry by chute, bucket, and pump methods with internal and surface vibration.

DESIGN OF MIX

Numerous investigators have reported on mixtures of concrete material in which the basic water-cement ratio is maintained while the gradings are changed to provide workability. Laboratory engineers have reported the benefits to be derived from making up combinations of sizes to economize on cement. Others, particularly reclamation engineers, have reported on adjusting size composition to minimize waste. In building the San Francisco-Oakland Bay Bridge these ideas were coordinated to a wide degree to solve the numerous problems in manufacturing concrete that arise where many different types of structural elements must be fabricated under conditions requiring a wide assortment of tools and materials.

However, with available materials, concrete proportions, tools and methods were developed to provide ample capacity, efficient operations and essential quality. Theoretical factors were not compromised more than practical operations usually compromise theoretical intent. In fact, the recognition of the compromises that regularly occur in field operations suggests means of making mixtures in which there is less compromise. It is believed that workability takes precedence over all other factors. The structure, design and specifications should be premised upon such mixtures. When this has been done, the contractor should be obliged to use methods and tools by means of which uniform concrete is manufactured.

A concrete mix containing plenty of mortar and no free water possesses certain advantages to the engineer and contractor. Mixtures of this type can be handled and placed under almost any condition. The necessity for accurate water control and uniform fabrication is

evident, but the necessity for careful grading of coarse aggregates is practically eliminated if weight batching is employed. Adequate strength is assured by controlling the water cement ratio. The only serious objection to this type of mixture is the increase in cement which might be required to decrease the water cement ratio to that of a mix requiring less water. However, this increase in cement need not be great, and many other conditions may more easily be brought under control by the use of this additional mortar.

Concrete batched and loaded on barges to be used under a great variety of conditions with a fixed water cement ratio, must be compounded uniformly from uniform materials. Concrete containing no free water, to be placed in watertight caissons, pier shafts, bridge seats, tunnels, reinforced elements and floor systems, must be put together properly. When loaded and out to sea, the entire cargo may go either into the structure or into the Bay.

Very little tolerance was allowed for aggregate sizes passing a $\frac{3}{8}$-in. and retained on a $\frac{3}{16}$-in. These sizes were avoided, resulting in a tendency to skip combined grading. Particular emphasis was placed upon the provision that the sand should contain approximately 30 per cent passing a 30 mesh sieve.

The concrete mixtures selected for use in the various units and elements of the Bay Bridge project were those in which the ratio of fine to coarse aggregate might be increased or diminished without increase in water, although, theoretically there is a limited variation in water due to changing the sand.

The subject of surface checking and incipient shrinkage cracks was given attention. This evil as it developed in the early period of construction was found to be associated with certain fine sands. Without any great change in slump, but by a change in sand grading, this surface checking disappeared.

CONCRETE DESIGNED FOR PUMPING

Properly designed concrete mixtures containing no free water can be readily pumped through pipe lines. The mortar should possess a maximum of cohesiveness to support the coarse aggregate column at the center of the pipe, and above all it should not contain an excess of intermediate sizes which interfere with the larger aggregate nesting together in this central section of the column. When these mixes are correctly designed, concrete made from rock or gravel up to a 3-in. maximum may be delivered easily into caissons or tunnel walls by pumping. The manufacturer developed a pugmill mixer and feeder for these pumps which effectively avoided segregation and tendency of the concrete to jam in the pipe line.

FIG. 20-23—S. F. O. B. BRIDGE

Fig. 20 (upper left) Placing concrete pedestal base suspension tower. Fig. 21 (upper right) Screeding concrete, base of suspension tower. Fig. 22 (lower left) Grinding surface for true bearing, suspension tower. Fig. 23 (lower right) Cleaning surface of tremie concrete after dewatering.

Six-sack tunnel concrete with a water cement ratio of 0.73 to 0.75 and a slump of 2-in. to 3-in. is being pumped 540 lin. ft. and lifted 60 ft. vertically through the pump line with entire success. The 28-day compressive strength of this concrete has ranged from 3800 to 4975 p.s.i., with an average strength of 4128 p.s.i. for 24 specimens. Five-sack Pumpcrete concrete in the caisson of Pier W-3 averaged 3685 p.s.i.

HANDLING TOOLS

The construction methods used in the Bay bridge units required that the concrete mixtures be transported by belt conveyors, cars, skips, chutes, bucket or tremie.

Concrete may be satisfactorily transported on belt conveyors, but its usefulness will depend upon what effective remedy is adopted to offset the evils of such means of conveyance. There are any number of cases where belt conveyors are very useful in solving the problems of *transporting* concrete, but belt conveyors like chute lines, cars and buggies, cannot always be used to *place* concrete. Transportation and placement are very different matters, although the problem of transporting and that of placing concrete are closely associated. If a properly designed concrete mixture is handled properly at transfer points, there is no technical reason for denying the use of any or all means of transportation. In fact, to insure low costs on projects of the magnitude and character of the Bay bridge, it is advisable to permit this freedom to the contractor, provided certain remedies for inherent evils of a transportation system are made effective.

If the complete batch from the mixer is delivered to the point of placement and possesses stability to insure uniformity of ingredients after transporting, that batch can be partially dumped at several adjacent points in the forms to make a homogeneous structural element. This is evident. Assuming all batches delivered from the mixer to be alike, the problems in transportation arise from the necessity of making batches which can be divided or strung out without losing any of the essential properties of the original mixture. The mix must be one which will not disintegrate beyond repair by means of remixing bell trays or hoppers at transfer points on belt conveyors or chutes or other transportation units. These remixing gadgets should be installed at every point in the transportation systems before vibrations or jolting cause disintegration beyond repair. If materials used in the mixture cannot be developed which possess properties to remedy this disintegration, the unit length of chutes or belts must be shortened or their use discontinued. It has been found necessary on this work to restrict the distance between

transfer points for chutes and belt conveyors, but materials have always been available to make concrete that could be moved by any system of transportation.

An interesting development was the fact that with the 6-sack concrete designed for underwater construction with a slump of 5 in. to 6 in. to secure the flow considered necessary for proper placement, it was possible to secure strengths in excess of 4000 p.s.i. in from 28 to 60 days as determined from cores cut from the top of the seal concrete of Pier W-2 after de-watering and also from cores cut from a block of the seal concrete broken off at an underwater depth of more than 170 ft. in Pier W-6, as shown by the following figures:

1. Cores cut from 6-sack underwater seal concrete top of Pier W-2:

10 days	1906 p. s. i.
34 days	3562 p. s. i.
60 days	4313 p. s. i.

2. Cores cut from 6-sack underwater seal concrete broken from base of Pier W-6:
34 days average strength 3954 p. s. i.

All concrete deposited under water by the bottom dump bucket method.

The base of Pier W-2 was placed from bedrock (cleaned with an hydraulic jet), at an average depth under water of 100 ft., in a continuous operation to within 12 ft. below mean low water. The top of this concrete mass was found, upon de-watering, to be covered with but a few inches of laitance. Cores were cut from the top after cleaning off about 2 in. of this laitance. The strengths were as given above.

It was evident from these tests and from inspection of the surface of the underwater concrete placed in other units that 5-in.-slump, 6-sack concrete of a properly designed mix can be deposited readily to any depth under water by means of the bottom dump bucket method without segregation and compressive strengths of 3500 to 4000 p.s.i. at 28 days can be readily secured.

VIBRATION

As an additional aid in securing density, the use of internal vibrators was specified for all structural concrete other than the roadway decks where the use of surface pavement vibrators was specified.

All of the contractors elected to use the Viber tamper type of internal vibrator for structural concrete. Both the electric and air powered types were used with excellent results. It was possible to handle concrete successfully with a lower water cement ratio and consequent greater density than usual in heavily reinforced structural work.

The placing of the roadway slabs has been started recently. The contractor is using two Jaeger-Lakewood automatic finishers with vibrator attachment—one finisher being approximately 30 ft. long to span the full width of the lower roadway and the other approximately

19 ft. long to span each one of the three lanes of the upper roadway.

All of the problems connected with this type of equipment and the proper design of the mix have not been entirely solved, however, 6-sack standard concrete with a water cement ratio of 0.72 and a slump of 2 in. is being placed satisfactorily in a heavily reinforced 6½-in. slab on the lower deck and 6½-sack light weight concrete with a water cement ratio of less than 0.70 in the upper roadway.

LIGHT WEIGHT CONCRETE

Considerable saving in the cost of the bridge was secured through the reduction in weight of members in portions of the structure by the use of light weight aggregate concrete in the upper roadway deck.

The design was based on concrete weighing not over 100 lb. per cu. ft. with a strength of not less than 3000 p.s.i. at 28 days with not more than 7 sacks of cement per cu. yd., the State to furnish the light weight aggregate.

These specifications were drawn following investigations and tests conducted over a period of a year by the Materials and Research Department.

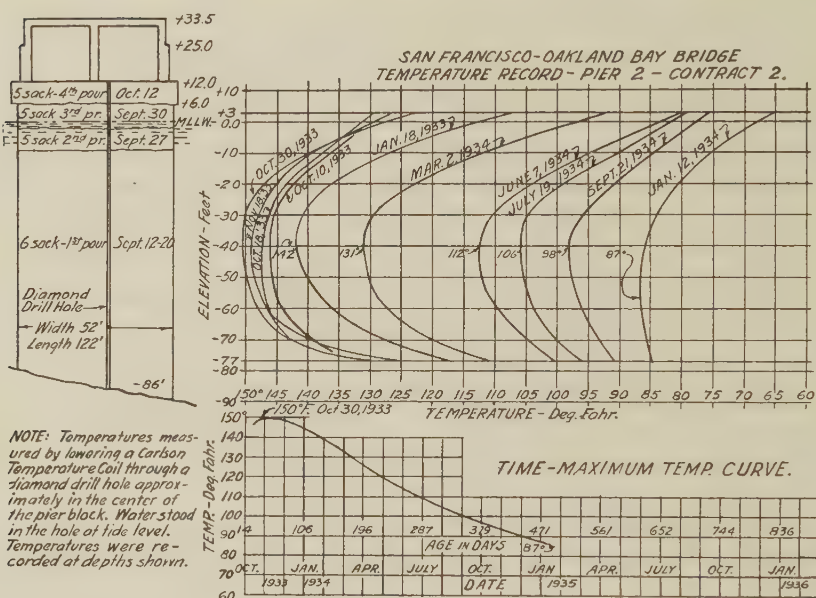
Early in 1934 a contract was let for the production of the coarse and fine light weight aggregate.

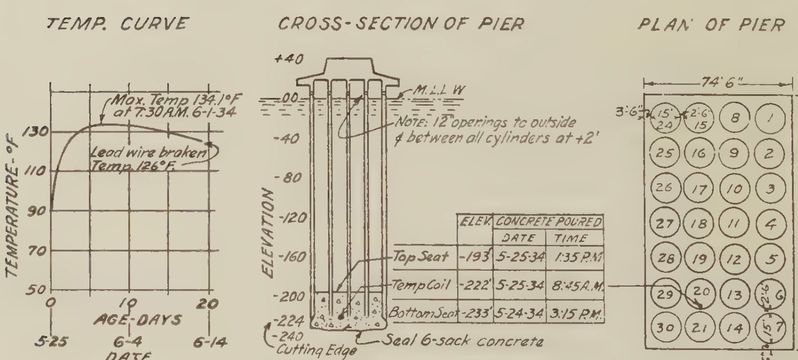
Paving the upper roadway was started the latter part of January, 1935.

Surface vibrators are being used for placing the concrete in the heavily reinforced 6-in. slab. Bottom forms have been stripped and in general show a perfect concrete surface without honeycomb. While there is very little tendency for the coarse light weight aggregate to work to the surface under vibration when the proper amount of water is used, the relatively heavy mortar readily works to the bottom of the slab under vibration, thus simplifying the under surface problem as compared with standard weight concrete. The result, however, is a deficiency of mortar and an excess of large rock in the surface, thereby increasing the finishing problems for a smooth riding surface. This difficulty is overcome by first screeding and vibrating the surface a quarter inch low and then adding sufficient standard sand mortar to make up the deficiency. The amount of mortar in the finished surface is not greater than that in standard pavement construction with standard aggregates.

TEMPERATURES OF UNDERWATER MASS CONCRETE

To secure some information as to temperatures developed during the setting of large masses of concrete deposited under water, Carlson temperature coils were installed in several of the units. One was





WATER TEMPERATURES

	Temp. °F
Outside ¢ close to caisson at high tide.	59.5
Inside wells before pouring seal. Ave at all depths.	61 0
Inside wells after pouring seal	63.0
Top of Concrete	65.0

FIG. 24 AND 25—TEMPERATURE RECORDS, CONCRETE PLACED IN SAN FRANCISCO-OAKLAND BAY BRIDGE

installed in the seal concrete of Pier W-3 approximately 224 ft. below mean low water. From similar tests conducted on other under-water units of the bridge, it was found that all mass concrete deposited under water in blocks 16 ft. or more in depth, can be expected to develop temperatures exceeding 130° F. in the center of the mass, even though placed in water of less than 60° F., also that even the surface of such under water masses can be expected to develop temperatures substantially higher than the surrounding water. In all probability, mass concrete deposited under such conditions develops practically normal strength at all periods.

SUMMARY

1. With raw materials similar to those readily available to practically all the cement mills in California, a high quality cement with less than 8 per cent C_3A can be manufactured without increase in cost sufficient to justify a premium price.

2. All high-iron low-C_3A cements manufactured for the Bay bridge readily pass A. S. T. M. requirements.

3. The low C_3A cement manufactured for the Bay bridge by all companies shows much greater resistance to sulphate action than the standard portland cements manufactured by the same companies.

4. The high silica cement used in some of the units of the Bay bridge developed at least normal strength and, under the sulphate tests, showed greater resistance to deterioration than the standard cement manufactured by the same company.

5. Following the general principle of concrete design outlined in this discussion, and by exercising rigid control over the production and shipment of the aggregate insofar as uniformity of grading is concerned, no serious difficulty was experienced in designing concrete mixtures suitable for placement under all of the placement conditions encountered on the Bay bridge including underwater concrete deposited in concrete seals to any depth, either by the tremie method in depths up to 50 ft., or by the bottom dump bucket method in depths up to 240+ ft. Concrete mixtures were designed for successful placement without segregation, by the chute, truck, car, wheelbarrow, or the Pumpcrete method.

6. It was possible by rigid control of the concrete mix to maintain a water cement ratio low enough, regardless of slump requirements based on method of placement, to result in a structural concrete averaging in excess of 4000 p.s.i. at 28 days with 6 sacks of cement per cu. yd. and a water cement ratio of .75 and a 5-sack concrete averaging over 3500 p.s.i. at 28 days with a water cement ratio not exceeding .85.

7. A high degree of uniformity in strengths was obtained, as determined by compression tests, on more than 2000 cylinders cast to date on the project. Uniform 3500 p.s.i. 5-sack concrete was fabricated suitable for placement under practically any structural design requirement, with the result that 5-sack concrete has been specified as standard for practically all structural units of the approach structures which have recently been let to contract.

8. There is no doubt that the uniformity in consistency and strength of the concrete produced on this project was made possible by the use of the automatic type of batching equipment specified, and that the specifying of this type of equipment is justified on any project large enough to absorb the initial cost.

9. By the use of automatic weighing equipment throughout it was possible to control the cement yield to within less than one per cent of theoretical for the entire project without constant changes in the mix based on discrepancies between successive placements. The cement yield seldom attained the maximum 2½ per cent variation fixed in the specifications.

10. Both internal vibration of structural concrete and surface vibration of heavily reinforced floor slabs makes it possible to place satisfactorily concrete of lower water cement ratio and greater density than hand placement methods.

11. Light weight concrete weighing not more than 100-105 lb. per cu. ft., with a strength of at least 3000 p.s.i., with not more than 7 sacks of cement per cu. yd. is being fabricated and satisfactorily placed in the 6-in. heavily reinforced upper roadway floor slab.

ORGANIZATION

The San Francisco-Oakland Bay bridge is being constructed under the California Toll Bridge Authority headed by Gov. Frank F. Merriam.

The construction is being handled by the San Francisco-Oakland Bay Bridge Division of the Department of Public Works, Earl Lee Kelly, Director.

C. H. Purcell, Chief of the California Division of Highways, is Chief Engineer of the Bay Bridge, Charles E. Andrew is Bridge Engineer and Glenn B. Woodruff, Engineer of Design. Board of Consulting Engineers, Ralph Modjeski, Chairman, Moran and Proctor, Leon S. Moisseiff, Charles Derleth, Jr., and H. J. Brunnier.

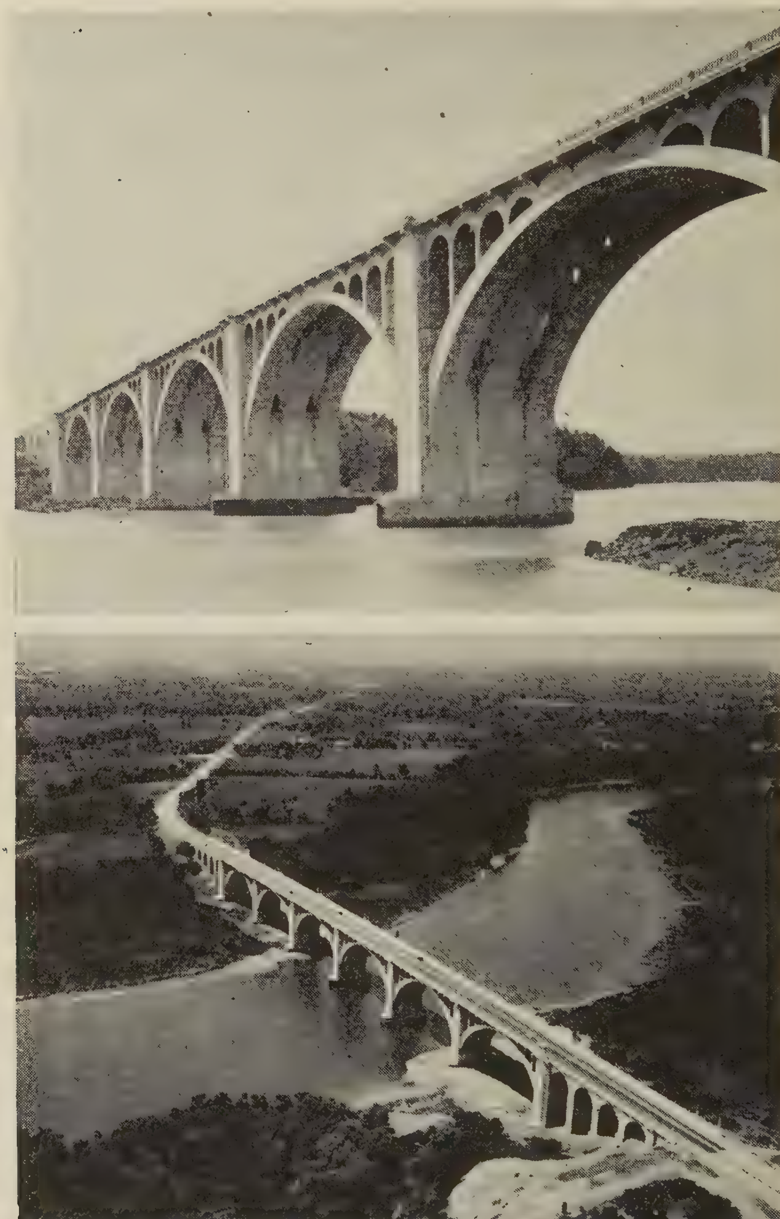
Resident Engineers: San Francisco Anchorage and Approaches, N. W. Reese; West Bay Piers and Suspension Spans, I. O. Jahlstrom; Yerba Buena Island Anchorage Tunnel and Approaches, H. Carter; East Bay Piers, Approaches and Superstructure, V. A. Endersby.

All local structural steel, cable wire, timber, piling, cement, concrete,

batching, paint and miscellaneous materials control tests are being handled by the Materials and Research Department of the California Division of Highways.

Stanley M. Hands, Associate Physical Testing Engineer, representing the Materials Department, is in charge of the concrete mix design and plant control.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for March-April 1936. Discussion should reach the Secretary by February 1, 1936.*



RARITAN RIVER BRIDGE—U. S. ROUTE NO. 25, NEAR NEW BRUNSWICK, N. J. (SEE "ARCHITECTURAL CONSIDERATIONS IN BRIDGE DESIGN")

Architectural Considerations in Bridge Design*

BY MORRIS GOODKIND†

Throughout the history of man, from the earliest records of his existence there have been evidences of his problems in overcoming obstructions to his progress in transportation and to this day some of the most ancient structures still in existence in one form or another are bridges. They form visible monuments to the civilization and culture of the periods which they represent and serve to tell part of the story of peoples of past eras in a very vivid manner.

Reaching the more developed stages of progress we find the picturesque type of Chinese construction adapted to their native characteristic requirements, followed by the monumental bridges and aqueducts of the Romans of massive stone masonry and continuing through the Mediaeval and Renaissance periods, each exhibiting their individual advances in the art and science of bridge building. The peculiarities of the cultures and artistic development of the civilizations of these periods are brought down to us by the architecture of their structures, and there can be little doubt of the enjoyment and interest which these respective peoples experienced not only in the use of, but also in the observance of these creations of their own peculiar art.

Until two decades ago the development of bridge engineering in America was confined almost entirely to the utilization of new and more efficient structural materials and the perfection of scientific principles of design to meet the requirements of the rapid growth and industrial progress of our country. There appeared to be little time or desire to consider the cultural welfare of the public with the result that with few exceptions we now look upon these structures as products of a period when our artistic sense must have been quite dormant.

Fortunately, this condition is gradually but surely being eliminated, and the principle factor in this transition is the expansion of the modern highway system throughout the country. More and more is the necessity for consideration of the appearance of bridges becoming apparent, but not until the bridge engineer recognizes that the general public demands beauty in all of his designs will he be considered to

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*Presented at the 31st annual convention, American Concrete Institute, New York, Feb. 19-21, 1935.

†Bridge Engineer, New Jersey State Highway Commission.

have given all the service rightfully expected of him. The revenue for public buildings is supplied by the same governmental agencies as State and County bridges and these agencies are very careful in the selection of their architectural service, materials and type of building so as to produce structures that are satisfactory from an artistic as well as an utilitarian viewpoint. Is there any reason why a bridge, which is probably used and seen by many more people, should not receive similar treatment?

Beautiful bridges at the gateway to a community are distinct assets, and set a basis on which the stranger forms his impression of the community. On rural highways an ugly structure will destroy the appearance of an otherwise attractive landscape. The public has always appreciated and praised the results obtained by successful esthetic treatment of its buildings and failure on the part of the bridge engineer to give due consideration to the appearance of his bridges reacts just as much to his discredit as faulty structural design.

Within the last few years we have witnessed the construction of a number of extensive bridge projects, by specially created bridge commissions whose organization included consulting boards of architects as well as engineers. The results obtained on these structures reflect credit almost universally on all concerned. Similar organizations were formed to supervise such developments as were undertaken by the Westchester County Park Commission. However, the sum total of all the bridges coming under such jurisdictions is a very small percentage of those which are continually under construction by the States, Counties and railroad systems of the country where the employment of consulting boards may not be feasible or necessary. The bridges in this category do not as a rule rank in importance with the monumental, record-breaking class nor do they require the specialized treatment of parkway structures. They do, however, call for proper expressions of artistic skill as well as structural stability and economic soundness, and it devolves upon the bridge engineer to acquire through training and experience, supplemented by an innate sense of beauty, the necessary qualifications to produce structures which are inspiring in esthetic treatment. Much has been said of the value of collaboration between the architect and the engineer in the creation of proper designs, but the architect without the proper background in structural analysis may be just as ineffective in producing the desired result as the engineer with no conception of the artistic.

Fortunately, the engineer is not bound by ancient architectural orders, forms, or styles, nor are there any fixed standards by which he must abide in the development of beauty in his bridge. Terms such

as Byzantine, Roman, Gothic, Renaissance, Doric or Corinthian may be entirely eliminated from his bridge vocabulary. The grace and charm of a structure will depend on the characteristics of the individual project and the engineer is in the position of exercising his natural talents and own artistic concepts in expressing the beautiful. The principles of esthetics are not subject to the same well-defined exactness as is applied to bridge design and it must be recognized at the outset that conceptions of artistic excellence are relative only. The structure is viewed by persons of various degrees of cultural development and native characteristics, so that one may create quite conflicting reactions to his efforts to obtain beautiful results. This difference of opinion is also evident among qualified critics of bridge architecture, which should emphasize to the engineer the possibilities which lie before him in producing structures of individuality in beauty just as he has pioneered in the development of the science of bridge design.

Of primary importance to a beautiful bridge is the requirement that it express the truth. It is built to carry with safety certain loads and its stability, and durability should be apparent to the lay observer in its lines, form and mass, as well as in the appropriate use of its materials. Each of its parts should exhibit a clear explanation of its purpose and the entire layout should bear a logical relation to the particular obstacle which it crosses. The masking of surfaces or members without visible justification or the use of lines foreign to the structural design leave a false impression and should be avoided as lacking artistic worth. Efforts to imitate materials such as stone by the use of scorings in concrete surfaces should be discouraged as being deceptive.

One of the essentials in a beautiful structure is the quality of simplicity. By applying careful thought to the outline of the entire bridge as well as to the lines and form of the component members, charm and grace may be incorporated in a strictly utilitarian design without involving expensive construction. This simplicity should extend to all of the elements, such as details, surfaces and choice of materials, so that the entire ensemble may express its prime function in a direct and pleasing manner. Meaningless ornamentation and embellishment merely help to distract the attention from the important features and serve to irritate rather than please the observer. Useless details and unsuitable decoration will detract from otherwise satisfactory structural lines and proportions. Simplicity may be attained in a structure regardless of the problems presented by the location, and by an earnest effort to abide by its principles the designer will have greatly improved the appearance of his bridge.

PETERS BROOK BRIDGE, ROUTE 31, SOMERVILLE, N. J.

Perhaps the most universally admired feature of a structure from an esthetic viewpoint is symmetry, and it probably is the most difficult to attain in perfection. Bridges are built to cross natural or man-made obstacles and they must of necessity conform to the conditions imposed by their location, but in all cases one should strive for as perfect a degree of symmetry as possible. This applies not only to the number, type and arrangement of spans, but also to the profile of the roadway and all features which should properly be parallel to it. By the use of an easy convex vertical curve we not only produce a dominant line of grace, but gain vertical under-clearance which is so often essential and avoid the appearance of sag and weakness that a straight line grade produces. This quality is more essential to bridges of short length where the eye of the observer may view the entire layout at a glance than where the bridge is of such length that lack of perfect symmetry can only be detected by an extended study of this feature. Crossings of wide streams usually require a long span over the main channel, which in turn fixes the required underclearance, and the location of the peak in the grade line. As is often the case, this natural channel is adjacent to one of the shore lines and the dominant span therefore is

located so as to make symmetry impossible. Similar conditions may result from the difference in character of the topography on both shores, from the number and type of additional obstacles that must be crossed or from foundation conditions which may control the number and length of spans. However the divergence from true symmetry should be in a regular manner, bearing in mind that the reason therefor must be made apparent to the observer so that the honesty of the layout may compensate somewhat for the lack of balance. Where no difficulties of a major character are presented toward gaining true symmetry, then, surely, this quality should be attained even at slightly increased cost since it is so elemental toward beauty. The location of the line of symmetry is worthy of consideration. For a single span crossing there can be but one choice. As the number of spans increases it is desirable that they be added symmetrically about the middle span, which becomes the focal point of the entire layout and appears to provide freedom for the flow of the stream where the current is greatest. However, when the number of spans becomes so great that the eye cannot at a glance detect the center-line of the bridge, this consideration does not bear such weight. Similarly in laying out such details as balustrade posts, brackets, lighting standards or spandrel columns, they should be spaced so as to avoid locating them on the center-line of a span. The natural reaction to such an arrangement is that of a concentration unnecessarily placed at the most critical point of the span.

To be pleasing to the beholder a bridge must possess harmony. It must be adapted to the landscape and be merged with its approaches so that it does not appear to intrude on the environment or dominate the natural scene. The ends of the bridge should not terminate abruptly without regard for the topography of the banks. A graceful transition from structure to approach may be obtained by extension of the wing walls into the natural ground lines, and in the case of a U abutment by continuing the balustrade. Similarly by recognizing the effects of the outline of the piers on the resistance to stream flow, and making full provision to reduce this resistance, there will naturally result a form which will harmonize with the current and be pleasing in appearance. Harmony of materials will lend dignity to a structure. The use of different materials for members serving the same functions will leave the observer perplexed and destroy the effect of other admirable features. Also the use of inappropriate materials, or camouflaged treatment of them, will create an impression of falsity. The objective of a complete structure should be one of unity. This involves a harmonious relation between the various parts of a span,

such as between the substructure and superstructure, between the arch rib or barrel and the spandrel walls or columns, between the lines of the columns or piers and those of the span they support, and between the floor and the balustrade or other details superimposed upon it. Where more than one span is involved harmony requires a similarity among them, or if such a condition is irrational for obvious reasons the break in similarity should be made well-defined by an intervening mass in a pier or pilaster. Under the classification of harmony we may also consider the requirement for continuity. Primarily the bridge is one small portion of the highway and as such should give the user thereof the feeling of safe and uninterrupted freedom of travel. He will naturally desire to view the landscape from the advantageous location of the bridge and obstructions in his line of sight will of course provoke him. Such an effort is not conducive to the enjoyment of the bridge. A deck type structure with no interference between the lines of the balustrades will serve to create the impression of continuity and security, while through structures, with unintelligible sections of the main carrying members protruding through the roadway, immediately direct attention to the existence of a hazard below and produce a feeling of anxiety as to the safety of the user.

The fourth of the architectural principles necessary for a pleasing structure is proportion. The purpose of each unit should be clearly defined and its mass should bear a logical relation to the work it is called upon to perform. This condition need not necessarily involve a disregard for economy since proper consideration in the design toward the requirements of economy and beauty will usually provide both satisfactorily. The manner in which superimposed loads are carried to columns, piers or abutments should be made obvious by a graceful and properly proportioned seat to indicate the adequacy of the transfer. The depth of superstructure should be in direct proportion to the span length it covers, for there is nothing so apparently inconsistent and unartistic to the observer as a shallow deck in a long span flanked by shorter spans of deeper dimensions even though in each case the structural analysis is correct. A bridge that is architecturally correct does not require that a designer explain to the general public, in whose interest it was built, the reasons for such discrepancies. The stability of the structure should be apparent, so that the user may travel over it with confidence.

The principles thus far discussed are basic in producing bridges of architectural merit. The degree of excellence attained will depend upon certain additional considerations, which may be termed of secondary importance but which nevertheless must be given careful

Raritan River Bridge, Route 31, at Somerville, N. J.

thought if charm, elegance and dignity are to be fully expressed by the construction. In this division we find the headings of lines, details and ornamentation, any of which may improve or destroy otherwise beautiful conceptions if they are nor properly treated.

With relatively few exceptions the main deficiency in bridges which were built during the pioneering period in this country was the lack of graceful lines. This was due in large measure to the materials available and the demand for the rapid development of means of communication. The ugly appearance of the harsh straight lines of the wooden, iron and steel bridges of this era is known to all. With the advances made in the analysis of suspension bridges and the rapid improvement in the design and construction methods of reinforced concrete the engineer has been afforded media for the use of the more distinctive curved lines. The inherent beauty of the circular segment, the ellipse or parabola will provide the grace that is so essential to the pleasing structure. It is recognized that arch construction is not always possible but by applying slightly curved lines to the bottom flanges of beams or to transitions between different types of construction some of the unfavorable reaction to the abrupt change will be avoided.

In the development of details proper weight must be given to their function, always bearing in mind however their relation to the main members of the structure. In an ideal layout the balustrade is practically the only feature of the bridge visible to the user and his impression is based largely upon its adequacy and beauty. Its dimensions should express strength without being cumbersome, and its design should be simple and permit an unobstructed view of the natural surroundings. Copings on piers, bridge seats and walls should be made bold enough to indicate character without distracting from the units of which they are a part. Overhangs in the form of bracketed construction and cornices may add distinction to the units supporting them, but by exaggeration of the overhang the shadows may obscure the more important features below them.

The question of ornamentation is subject to the most divergent views on the part of architects and engineers alike if one is to judge by bridges constructed within the past decade and by the statements of critics. The following quotations are used as examples: "Wood or steel forms are necessary, in most cases, to hold the concrete in the shape it is to have finally. Why not, therefore, permit the marks of these forms to be exposed since they tell the story of the mode of construction? More often these markings look better than a treated surface if the construction of the forms is the subject of careful planning." Another authority writes in discussing the virtues of European bridges, "A structure is never seen left with its rough, unsightly construction finish showing the imprint of forms." Again, "Neither should stone facings be used in a concrete or reinforced concrete bridge, but the entire structure should be truth-telling in every respect." We are all familiar with the stone faced concrete bridges which have been constructed in the last few years and which have been pronounced beautiful, but perhaps conditions make fixed rulings inadvisable in specific instances.

The purpose of ornamentation should be to accentuate the function of the individual member to which it is applied in relation to other component parts of the bridge, and to the entire bridge. By proper detailing of decorative treatment stress may be laid on the relative importance of the various lines in the structural design so that an instructive as well as a pleasing picture is presented to the observer. Piers supporting columns should be defined at their junction by a coping. The columns in turn should be provided with appropriate seat detail if the deck simply rests on it. On the other hand if the column and superstructure form a continuous frame this phase should be clearly emphasized. The arch barrel and spandrel wall may be

made distinctive by slightly recessing the latter, by different surface treatment or by panelling. The spandrel wall and balustrade may in turn be defined by a cornice parallel to the roadway always bearing in mind the danger of making this cornice or overhang overbearing.

There is one other justification for ornamentation. That is its use in improving large flat surfaces which would otherwise be uninteresting and monotonous. It is the practice of railroads operating over electrified systems to require solid balustrades, 6 ft. 6 in. high to avoid interference with their catenary lines. In such cases, ornamentation is essential to reduce the wearisome effect on the observer. Similarly some slight treatment of long wing walls will add interest to its surface. One must always be careful however that the amount and proportion of ornamentation does not overemphasize it in relation to the member which it is intended to decorate.

It would of course present an almost insurmountable problem to conform with all of the principles discussed heretofore with some of the structural materials available to the bridge engineer. Through the medium of concrete, however, he is in a position to apply them practically and economically, and may treat all bridges, regardless of size or importance as means of expressing dignity and charm. Mass, which is so essential to proper proportion, is an inherent quality of concrete. Graceful lines may be readily obtained and the appropriate form becomes a simple matter in the field. It is an ideal medium for ornamentation and both in design and construction it is readily adapted to the desires of the engineer.

It is to be noted that all of the principles and considerations requisite to produce an artistic conception are interdependent. It therefore is necessary that the complete layout, including details, be carefully studied at the very inception of the design in order that the project may satisfy the unity required in an esthetic creation. On more important bridges this may necessitate perspective drawings of the entire structure so that the suitability of the details may be judged in its relation to the more important features and the balance of all of the parts may be properly adjusted. Photographs of the site itself may advantageously be incorporated in these studies to assist in harmonizing the structure with its environment.

The esthetic excellence of a bridge is not subject to the same positive analysis as is the structural design, but if the engineer with good taste and judgment, will apply himself as thoroughly to acquiring a conception of the artistic as he has to the perfection of his scientific theory, he will then be in a position to contribute to the cultural

progress of his time. The bridge has always been an index of the development of its particular age and should now be, as ever, characteristic of the standards of our time. The bridge designer has an enviable opportunity for achievement in bridge architecture and for creating an era of beauty that will serve as a distinct contribution to civilization.

For such discussion of this paper as may develop readers are referred to the March-April 1936 issue of this JOURNAL. *Discussion should be available to the Secretary by Feb. 1, 1936.*

Inspection of Concrete

The two following papers, presented at the 31st Annual Convention, New York, Feb. 19-21, 1935, consider various aspects of the problem of adequate concrete inspection. A. Burton Cohen and R. B. Young are agreed on the importance of inspection as an often-neglected link in the chain of responsibility which should achieve in the structure the quality anticipated in the design. Insofar as their views are divergent, their solutions of the problem are in the kind of training and experience which should be required of inspectors. Discussion of these two papers will be consolidated in the March-April, 1936 issue of this JOURNAL. *See note page 50*—EDITOR

Supervision and Inspection of Concrete*

BY A. BURTON COHEN†

MEMBER AMERICAN CONCRETE INSTITUTE

There are three distinct branches of structural engineering. Each is characterized by a material preponderantly used in its particular development. The materials are timber, structural steel and concrete. Each branch may be divided into three major subjects: research, design and construction and these subjects further subdivided into a number of interdependent considerations. The perfection of the development in each structural specialty may be measured by the degree in which the three major subjects may be closely correlated.

It is not surprising to find in the branch of structural engineering dealing with concrete and reinforced concrete, that research, design and construction have not been correlated to the fullest possible extent. Such prodigious development has taken place in a very short time that it has been impossible to translate all the findings of research in terms of design and construction. It is evident, too, that a higher degree of correlation is required in concrete work than in timber or structural steel.

Due to the fact that concrete is brought to a plastic condition and can be molded to fit any particular structural shape, the possibilities of creative art are unlimited and therefore varying results are to be expected. The most effective work in concrete is performed at the site of the structure under a wide range of temperature changes while other structural materials are made under cover and temperature control. Further conditions affecting construction will be mentioned later. The premise is here made that the major subject of construction in concrete work has not kept pace with the advancement in research and design and that the lack of uniformity concerns supervision and inspection.

A remarkable development in concrete work has taken place in this country since 1900. Only 35 years ago American engineers began to think seriously in concrete. Mention cannot be made of the early

*Presented at the 31st Annual Convention, American Concrete Institute, New York, Feb. 19-21, 1935.

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development or beginnings of concrete without referring with pride to the achievements of the workers in research.

For many years at meetings of the Institute the results of certain Watertown Arsenal tests were quoted and discussed. G. A. Kimball, Chief Engineer of the Boston Elevated Railroad, was responsible for these tests, reported in 1899. The echo has finally died away.

Prof. W. K. Hatt made a series of tests of reinforced concrete beams in 1902, determined the modulus of elasticity of concrete in 1904, and considered the time element of loading in 1907. In 1904, Prof. F. E. Turneaure at the University of Wisconsin made further tests on beams and, collaborating with Prof. E. R. Maurer made an excellent contribution in a text book covering principles of reinforced concrete. Prof. A. N. Talbot at the University of Illinois was also investigating the strength of beams in 1904 and on columns in 1906. Prof. M. O. Withey of the University of Wisconsin, tested beams in 1906, columns in 1909 and later carried on a 10-year test to determine the effect of age and curing conditions on the strength of concrete. In 1905-07 Prof. Ira Woolson thought about thermal conductivity of concrete mixtures and the effect of heat upon the strength and elastic properties. Sanford E. Thompson made valuable research pertaining to the consistency of concrete before 1906 and Buell and Hill were responsible for a very fine text book on concrete in that year. As far back as 1889 Prof. Baker included in the first edition of "Masonry Construction" an article on plain concrete which in the edition of 1907 was developed into several chapters on plain and reinforced concrete. Fuller and Thompson developed laws of proportioning.

These are a few of the pioneers who recognized great potentialities in structural concrete, devoted a life time toward its development, and from their early work laid the foundations and the spirit of the great research work which followed. They did not find it inconsistent to crush good natural stone to make a so-called artificial stone, because they saw a wide application ahead in the flexural possibilities of embedding steel reinforcement in the mass to take tensile strains. They were not hidebound by precedents.

Then came another group of investigators. F. R. McMillan in 1915 reported the shrinkage and time effects in concrete, made other investigations and is now Director of Research of the Portland Cement Association. A. T. Goldbeck from 1910 to 1916 deliberated over the expansion and contraction of concrete while hardening and made one of the most valuable contributions affecting design through his tests to determine the distribution of concentrated loads on a series of beams and slabs. These tests proved that many of the reinforced

concrete railroad bridges of that day were 40 to 60 per cent overdesigned and a similar condition prevailed in the design of highway structures. A. B. McDaniel carried on a series of tests to determine the influence of temperature on the strength of concrete in 1916. All concrete engineers should be familiar with every detail of these tests since they are most valuable as a guide in the removal of forms in all degrees of temperature. Bates, Philips and Wig made valuable contributions in research to determine the action of alkali water and sea water which included an examination of structures all along the seaboard. In 1917 A. R. Lord tested flat-slab girderless floors of large buildings and was largely instrumental in the development of this very important use of reinforced concrete.

More lately Duff Abrams in an exhaustive and most comprehensive investigation, established a law that fixed a definite relation between the strength of concrete and the ratio of the water content to the cement. The work of Duff Abrams is the most outstanding achievement in the matter of proportioning or designing concrete mixtures and has stabilized the art. Professor Slater was another outstanding investigator from 1907 until his untimely death in 1931. Professors Richart and Scofield collaborated with Professor Slater in tests on bond resistance, effect of electrolysis. Professors Richart and Lyse later collaborated with Slater on the A. C. I. column tests at Lehigh University and the University of Illinois. Hool and Johnson, Taylor and Thompson, Smulski, Professor Sutherland and Walter M. Clifford, C. C. Williams made valuable contributions in books on concrete, plain and reinforced.

And recently research workers are splitting the atoms of cement, so to speak. They are combining and recombining the four principal compounds of cement in various proportions for the purpose of controlling or reducing to a practical minimum, the heat generated in the setting of cement and thus reduce volume changes or shrinkage and the internal stresses resulting therefrom. This is the type of research work guided by Professor Davis of the University of California, which was carried on with reference to the mass concrete work of the Boulder Dam, reported at the 1933 convention of the Institute.

One could continue indefinitely to give just the high points of a most intensive endeavor in research work in this country, from which regulations have been established which control the making of concrete in a manner as scientific and as effective as the treatment of iron to produce steel of various degrees of hardness.

The American Concrete Institute has been a most outstanding factor in this fine development of concrete work. No problem is held in

abeyance. Search is made for information bearing on the intracacies of the art as they develop month by month.

The character of design to accomplish a given purpose may vary in many ways, depending upon the whims, the imaginative or creative power of the designer combined with his knowledge of research and construction. Included in research work is that invaluable review of what has been accomplished—a review of experience. The finished structure to the layman's eye and even to the more experienced does not always reveal the difficulties and the hazards involved in construction, and the degree to which the structural shapes were proportioned or molded to give the most economical and practical application of the materials. The writer attempted to emphasize the importance of these considerations in a paper presented at the 1926 convention of the Institute.*

Concrete design in America has advanced to a point where it can be compared favorably with European practice. Continuous spans and rigid frames, long span arches of graceful contour and a fine development in flat-slab girderless floor construction.

But, assume that the design is based upon the best procedure established by years of research and experience, there remains that last important step of building the structure. In this there are two responsibilities—that of the engineer in supervision and the contractor's in the performance of the work. The effectiveness of the contractor's work depends in a large measure upon the manner in which the engineer discharges his responsibilities. For the present, in this discussion of upervision and inspection the contractor's work is irrelevant.

As previously stated it is more difficult to correlate construction with research and design in concrete work than in any other structural activity because of the variables in construction caused by temperature changes. In addition, a great amount of inspection of materials must be made at the site, different degrees of strength and workability of the concrete must be obtained, there are numerous methods of placing the concrete, the curing and the protection are important, the form construction must be adequate and of good workmanship to secure the required surface finish of exposed concrete, the reinforcing steel must be placed in the correct theoretical position, and there are so many other minor details to be assimilated. In the responsibility of supervising all this work, the engineer carries many functions that are included, for example, in allied manufacturing agencies regulating and controlling construction work in structural steel. There are the

*"Correlated Considerations in the Design and Construction of Concrete Bridges"—making its author a Wason medalist.—Editor

similar responsibilities of the experienced roller, the inspection of the steel after rolling, and the fabrication in the shops.

There are many ways in which concrete work is supervised and carried on. A resident engineer may be in charge of the entire job assisted by an inspector, who may be a preferred civil service appointee, a chainman, or a rodman of long standing or any other personality of little experience, any one of whom may be considered by the contractor and even the owner, as a necessary evil. The frailities of inexperience and incompetence of this socalled inspector, with whom workers in concrete are familiar, will not be further discussed. It is shocking, however, to find big users of concrete treating inspection of concrete construction with any degree of indifference.

The functions of the resident engineer are manifold. Usually he has all the work he can effectively control in directing the general engineering work. At the time that he attempts to inspect materials, form construction, the concrete in the making and a multitude of other regulations, he may be interrupted by some seemingly more important function of general engineering and the concrete work is without supervision. It is then delegated to the incompetent inspector previously mentioned.

Despite the fact that the resident engineer may be experienced in general construction work, he may not have carefully followed the current development in concrete and he may not be familiar with or have forgotten theoretical considerations of design. Is he thoroughly interested in concrete work to the point of making a speciality of it? If he is not, the work lacks proper supervision.

It is not to be inferred from this one example of control that all concrete supervision is not reasonably well developed. There are many organizations that have developed the proper grade of supervision and control. Nevertheless the inexperienced inspector is still on the job and the character of the supervision may be found to be inadequate.

Ask a group of engineers, of long training in concrete work, what the qualifications of an inspector should be. It has been found that a summary of these qualifications reveals an overwhelming preference for the technically trained engineer, or his equivalent, the requirement of an intensive training in the fundamentals of all three major subjects of research, design and construction. This position may be emphasized by quoting from the works of a well known writer, as follows: "The position which the one part occupies in relation to the other parts cannot be rightly conceived unless there is some conception of the whole in its distribution as well as in amount." Qualifications of the inspector pile up until we have all the requisites of a specialist.

Here is a big field for the college man of the right temperament, personality and aptitude. Other industries through their personnel departments, assisted by similar departments at the universities, find college men to meet their requirements and a fertile field for such choice is open to the concrete industry.

There is a place on the average job for both a resident engineer and an engineer-inspector with whatever assistants the latter may need depending upon the size of the job. Confine the duties of the resident engineer to the general supervision of the work and the engineer-inspector to the specific duties dealing with the inspection of all materials, appliances and regulations affecting the concrete work. The engineer-inspector is the technically trained man specified after he has served his apprenticeship as an assistant.

What are the prospects of this technically trained assistant, or his equivalent? As good, if not better than his prospects would be in any other field. This statement is made with no regard to the present economic condition. Construction work is extremely interesting and absorbing, and a wide field is open to this young assistant. He may turn to design, after several years in the field. This would tend to improve the character of design since he would be better able to adjust his design to meet the practical considerations of construction. He may continue on through the duties of an engineer-inspector to a superintendant of construction and further to the control of the contracting field, to the end that a better type of contractor shall be available.

The question of dividing the relative position of authority between the resident engineer and the engineer-inspector is a matter of organization. The construction field in concrete needs men trained in the art, and the compensation for services rendered should be in line with the importance of their work and comparable to the compensation of the trained engineer responsible for the quality of materials and workmanship in other structural activities. There is a lack of quality and compensation in our present scheme of supervision of construction.

In the engineer-inspector there would be created the spirit of the research worker with a full complement of practical ideas—a competent engineer in the field concentrating specifically on proper interpretation, at the inception of every operation of the correct intent of the plans and specifications, and to secure with ease and certainty the sound concrete that years of intensive efforts of the research workers have made possible. It is the belated interpretations of the plans and specifications that cause the friction on the job. Lead the contractor in the right direction with knowledge and forethought, and he will take it with a smile.

Inspection*

BY R. B. YOUNG†

MEMBER AMERICAN CONCRETE INSTITUTE

In its basic principles, the requirements for adequate inspection are not greatly different, whether one is dealing with mechanical or electrical equipment, with the materials used in engineering construction such as concrete and steel or whether such inspection is done in shop, foundry or field.

A good inspector is a man who knows as much or more about the work he is supervising than the men doing that work. He must know how things are done as well as why things are done, and of the two it is more important that he know how than why. A good inspector must also be a man of character, fair in all his dealings, and liked and respected by the men with whom he is working. He must be able to get what he wants and make the other fellow "like it."

The more knowledge an inspector possesses about the work he is doing, the better able he will be to cope with the many problems that he will meet daily, for no specification, however complete, will begin to cover all situations that arise during the course of the average job. Hence, the engineer-inspector visioned by Mr. Cohen is ideal, provided of course, that he possesses the other qualifications.

I have employed engineers as inspectors and as a class they have not proved themselves as good as those not so well trained technically. The reason is simply that the engineer considers an inspector's job to be a junior position, an excellent place to get experience but nothing more. Consequently, your young engineer, if he is ambitious, is not satisfied to make inspection his life work, will not set himself wholeheartedly to learning the job, and therefore seldom becomes the equal of a lesser educated man, whose ambition does not aspire to such great heights.

This being so, then before any general adoption of the use of experienced engineers as inspectors can occur, employing engineers and architects must become convinced of the necessity for engineering

*Presented at the 31st Annual Convention, New York, Feb. 19-21, 1935.
†Testing Engineer, Hydro Electric Power Commission of Ontario, Toronto.

inspection, and raise the status of the job by paying salaries that will attract the desired type of men. This can only come about gradually, for at present not one employer in a hundred considers this necessary.

I have no quarrel with Mr. Cohen's proposals; they are both sound and desirable. But except where work is under the supervision of men holding his views, inspection has still to be done under the conditions of the present and how, in these circumstances, can competent inspection be obtained. I would like to offer as one solution to this problem, the methods used successfully over a period of many years by the Hydro Electric Power Commission of Ontario.

The Commission has found that men of foreman or superintendent grade make excellent inspectors if they have the right personality, and its whole inspection organization is built around this type of man. These men must be capable of actually directing work similar to that which they are called upon to inspect. Thus they are mature men, of experience and character, who have already achieved a measure of success in their profession; and hence command the respect of those with whom they have to deal.

This type of man is much superior to a young and inexperienced engineer, no matter how good a man the latter may be. Being older, he takes his job more seriously; he has a greater sense of responsibility, better judgment and he has learned to be more diplomatic in dealing with others.

Another advantage possessed by this type of inspector is that he does not have to overcome that inborn hostility which the average workman has toward those better educated. The friendliness of the men, and I include the foreman here, is a great asset to an inspector, for their co-operation is more important in getting a good job than that of the management.

The successful use of this type of inspector requires the recognition of certain fundamentals. No good man will be happy in a job in which he cannot take some pride; he wants to feel that his work is of real value. Because of this, it is necessary that inspection be treated as essential work by those in charge; that the job be dignified by recognition of its importance.

In a large organization, another necessity to a satisfied personnel is that, as far as possible, these men should not be competing with engineers for better positions; that is, all the better jobs shouldn't be filled by engineers. Of course, some competition cannot be helped but there should be a line of promotion available to them distinct from that available to any engineers doing inspection work. Engineers must be used for certain types of inspection work because of the

engineering character of the requirements, but on the other hand, certain better paid senior positions in the inspection field require only the addition of executive ability to the other qualifications, and preference to these should be given to the non-technical man as a reward for ability and service.

Let us now deal more specifically with the inspection of concrete work. Much of the unsatisfactory concrete we see about us is due to poor materials and workmanship, particularly the latter, and the difference between a poor and a good job is usually only a matter of the complete supervision of the details of proportioning, handling, placing, and curing. Hence, on any concrete job, there should be a man who knows how to look after such work and has the time necessary to do it properly.

A job like this does not require a concrete expert or a trained engineer, but the man should have a good working knowledge of the properties of concrete and of the fundamental requirements of good concreting practice. He also needs to know something about the testing of concrete and concrete materials, and the interpretation of test data in these fields.

The experience of the author indicates that the best place for a future inspector to get this knowledge of concrete is in a laboratory. In no other place does he learn so convincingly the laws governing the production of concrete. It is easier for a laboratory man to acquire field experience than for a field man to learn laboratory methods. The former will carry into the field the scientific viewpoint of the laboratory and any narrowness of outlook that he may have when he leaves there, will be corrected by his new environment; while the latter finds it hard to overcome his initial contempt for things theoretical.

It is a fundamental requirement of good management that responsibility and authority go hand in hand, and if a subordinate is given one, he must have the other. It is also a fact that responsibility can only be partly delegated because in the last analysis, some one man is finally responsible for the carrying out of any work and while he can delegate authority, he cannot escape responsibility for the acts of his subordinates.

This being the case, the degree of authority that can be given to any subordinate must be based on that individual's particular qualifications, which makes it difficult, if not impossible, to standardize inspection jobs even in like work.

Obviously, a man will need the most supervision in those matters in which he is least experienced, and a good boss will supply this supervision, but there are certain types of knowledge whose lack cannot

be completely overcome by supervision and many matters of workmanship are in this category. They cannot be taught except by example. Again, questions of workmanship when they arise cannot be postponed but must be solved immediately; as for example, the sequence to be followed in placing a complicated section of reinforced concrete. But generally, engineering problems will permit of some delay in their solution and usually are subject to review by the engineer in charge. For these reasons, it seems more important on a concrete job to emphasize the inspection function and sacrifice the engineering if there is a question of choice between the two types of experience.

The necessity for concrete inspection is independent of the size of the job, rather it depends on its importance; but the manner of providing inspection will differ. The big job is easier to organize than the smaller, but it is equally important that each have supervision of the kind described, viz., a man closely associated with it who knows concrete. On the bigger jobs, one man of the proper qualifications can be put in charge of a group of very average inspectors and by their proper supervision and training, get them to function satisfactorily. Surprisingly good results have been obtained in this way with mediocre men.

It is also a requirement of good management that each job, large or small, have a recognized head and in carrying out the field inspection of concrete, this can best be provided by making the inspector responsible to the resident engineer. The Hydro Electric Power Commission has found this system works well and on larger operations where the resident engineer has several assistants, the concrete inspector ranks in authority with the principal engineering assistant, and is paid a salary commensurate with his position, even though in many cases he is not an engineer by training. On the Commission's smaller jobs, the concrete inspector will outrank other assistants since his responsibility is much greater than theirs. On very small jobs it occasionally happens that the concrete work is the most important element and here a concrete inspector is often put in complete charge and may act as both inspector and foreman.

The one-man job is the hardest to provide for. You can't expect one man to act as engineer, clerk of works and inspector, and satisfactorily fill all three positions; and this is equally true whatever the training of the man. Usually when this is attempted, the engineering and inspection work has to be neglected to take care of the clerical detail and the job receives little real supervision. In building construction, it has been my observation that few reinforced concrete jobs were so small that their proper supervision would not require two men,

a resident engineer and an assistant. But many of these jobs won't have even a clerk of works, supervision taking the form of periodic visits by the architect or engineer and a few tests made by an inspection company. The result is that the contractor does practically as he pleases, only following the specifications in a general way.

Some engineers and architects provide for the inspection of their concrete work through the services offered by commercial testing laboratories. This is satisfactory if the laboratory employ experienced men and the mistake is not made of having the laboratories' inspector entirely independent of the resident engineer. As I have said before, one-man responsibility is a fundamental requirement of good management, and if the resident engineer has no authority over the inspection of his own concrete, he naturally is not going to accept responsibility for any trouble that develops. The system is very conducive to "buck passing."

My purpose in presenting this paper is not to advocate a system of concrete inspection or start an argument as the program might suggest, but merely to present my personal views of a subject that is given far too little thought by most users of concrete.

For such discussion of the papers by Messrs Cohen and Young as may develop, readers are referred to the JOURNAL *for March-April, 1936. Discussion should be available to the Secretary by February 1, 1936.*

Studies of High Pressure Steam Curing of Tamped Hollow Concrete Block*

BY CARL A. MENZEL†
MEMBER AMERICAN CONCRETE INSTITUTE

THE RESULTS of experiments on small specimens reported in the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE for Nov.-Dec., 1934,[1] indicated that by curing in high pressure steam, concrete could be obtained within 1 or 2 days after molding having substantially higher strength and lower volume change than concrete cured moist for 28 days or more at normal temperatures. They indicated excellent resistance of the steam-cured concrete to freezing and thawing and to the action of solutions of sodium and magnesium sulfate. They showed also that during exposure to high pressure steam the somewhat soluble calcium hydroxide or lime which results from the hydration of cement reacts with finely divided silica in the mix to form a fairly insoluble compound which not only contributes to the permanent strength and denseness of the hardening paste, but practically eliminates leaching and efflorescence. The steam-cured concrete was usually of substantially lighter color and considerably drier than moist-cured concrete at the end of the curing period.

To obtain further information of value to the products manufacturers the experiments on small specimens were supplemented by tests on the steam curing of large concrete specimens. These further tests were planned to provide a basis for definite recommendations for the steam curing of a wide variety of concrete products in the most effective and economical manner. This paper presents the results of that part of the study dealing with tamped hollow concrete masonry units.

Since the fundamental considerations in the steam curing of portland cement mixtures were thoroughly covered in the earlier report, this study was limited to a much narrower scope. One of the principal developments from the earlier tests was the important influence of

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[1]"Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete," by Carl A. Menzel, Am. Concrete Inst., *Proceedings*, Vol. 31, p. 125.

the amount and character of siliceous material in the mix. It was definitely shown that the full possibilities of high pressure steam curing for the rich, dense mortars could best be realized when finely divided silica was present either as a component part of the cementing material or as part of the fine aggregate. These results from the earlier tests must be taken into consideration in evaluating the results of the present investigation, but it should be pointed out that the influence of silica is of less relative importance in the very lean mixes used in hollow concrete block manufacture.

SUMMARY AND CONCLUSIONS

For the convenience of the reader, the principal conclusions and recommendations precede the description and details of the tests.

1. It is entirely feasible to cure tamped hollow concrete masonry units in saturated steam at 350° F. ready for delivery and use within 24 hr. after molding.

2. The steaming treatment should begin not earlier than 5 hr. after molding, but may be applied at any convenient time thereafter.

3. The steaming cycle should provide for a full 8 hr. exposure to saturated steam at 350° F. and a gage pressure of 120 p.s.i. This period of constant temperature and pressure should be preceded by a heating period during which the temperature in the steam chamber rises gradually to 350° F., and followed by a cooling period during which the temperature and pressure is gradually lowered. The length of these periods may be varied considerably to meet production needs, but until experience in practice demonstrates otherwise it appears that the heating period should be a full 5 hr. and the cooling period as long as possible but not less than 5 hr.

4. For all of the mixtures and materials tried in these tests the steam curing was completed without change in shape or dimensions, or the development of cracks in the hollow units. The units were comparatively dry and substantially lighter in color than the moist-cured product, and gave a clear ring when tapped with a hammer. Their compressive strengths compared favorably with those of units cured moist for 5 days and in air for 55 days at 70° F. Aside from a tendency towards increased brittleness with cinders and Haydite, the steam-cured units appeared on the whole to be superior to the moist-cured product.

5. A special advantage which seems to accrue from the steaming treatment is the assurance it gives that the units will not pop or spall in service due to the swelling of unsound aggregate particles or other causes. Exposure to the high pressure steam is in a sense an accelerated test for disruptive characteristics.

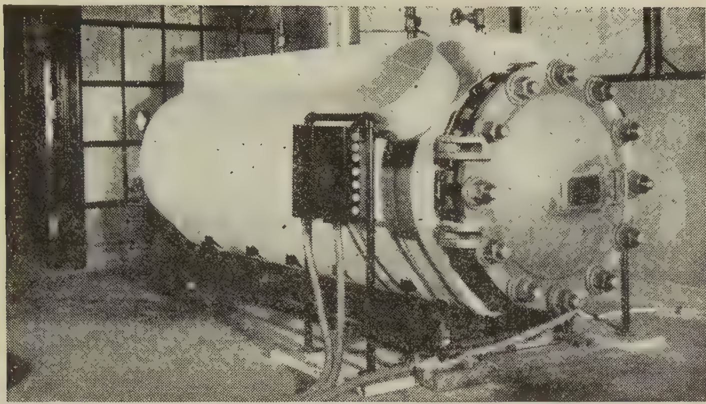

FIG. 1—GENERAL VIEW OF INSULATED CYLINDER USED IN HIGH-PRESSURE STEAM-CURING STUDIES

The cylinder, 30 in. diameter by 10 ft. long inside, is designed for working pressures up to 200 p. s. i. Saturated steam is generated by heating 6 in. of water in the bottom of the cylinder with a gas flame. The rate of temperature rise during the heating period is controlled by adjusting the gas flow. The desired pressure is automatically maintained during the constant-temperature and constant-pressure period by a pressure controller connected to the gas burners.

6. The shrinkage of steam-cured concrete in drying from a saturated condition to equilibrium with air in a heated building has been shown by the previous tests to be at least 50 per cent less than that of moist-cured concrete.

7. Such comparisons as can be made with the earlier tests referred to above indicate that the strength of hollow masonry units, like those used in these tests, when steam cured is influenced by the amount and character of the siliceous material in the mix, but to a much less degree than was shown for the richer and denser mortars in the earlier tests.

GENERAL DESCRIPTION

The steaming chamber used is described and illustrated in Fig. 1. The tests, which were confined to hollow concrete block units, were arranged to determine the influence of such factors as type of aggregate, mix, character of steaming treatment, and age of block at the beginning of the steaming period, on the strength and other properties.

Block. The one design used was the nominal 8 x 8 x 16-in., 3-core block so widely manufactured by concrete products plants. It has face shells from 1½ to 1⅝ in. thick, webs from $1\frac{1}{16}$ to 1¼ in. thick, and an average core area comprising 37.5 per cent of the gross area of 126 sq. in.

Aggregate. The three aggregates used were Elgin sand and gravel, Haydite, and soft coal cinders graded nominally from 0 to ⅜ in. In one of the later series of tests 0 to ⅜ in. crushed limestone was also used.

The larger gravel particles, between the No. 4 sieve and ⅜-in. size, were mainly calcareous and contained only 18 per cent of siliceous material. The silica content of the particles below the No. 4 sieve, however, was much higher and increased markedly with decrease in size as follows: No. 8-4, 26 per cent; No. 14-8, 32 per cent; No. 28-14, 45 per cent; No. 48-28, 65 per cent; and 0-No. 48, over 95 per cent.

The Haydite was typical of the light, porous aggregate material prepared by burning shale to incipient fusion and crushing and grading the resulting clinker. The material used was manufactured at the Danville, Ill., plant of the Western Brick Co.

The cinders used were obtained from the Crawford Ave. power plant of the Commonwealth Edison Co., Chicago, and the limestone was a dolomitic limestone from the Chicago area.

Mixes. Usually three mixes were used, which may be designated as lean, medium, and rich, depending on the relative quantity of cementitious material in the block.

Steaming Cycles. Two steaming cycles were commonly used. One, a 22-hr. cycle, permitted steaming only one load every 24 hr.; the other, a 12-hr. cycle, permitted the steaming of two loads every 24 hr. A third intermediate 18-hr. cycle was used in some of the later tests when information on the results with the first two cycles was available. These cycles were divided as follows:

Cycle	Heating Period In Chamber	Period at Constant Temp. of 350°F. (120 lb. Gage Pressure)	Cooling Period In Chamber	Temperature When Removed From Chamber
22 hr.	5 hr.	12 hr.	5 hr.	180°F.
12 hr.	3 hr.	8½ hr.	20 min.	Appr. 200°F.
18 hr.	3 hr.	8 hr.	5 hr.	180°F.

Steaming Age. The block were steamed at ages of approximately 1, 4, 6, 8, 20 and 76 hr. after molding.

Molding of Block. The block were made with a power tamping machine (Anchor junior power-tamper, hand stripper block machine).

The cement consisted of the laboratory mixture of equal parts of 4 brands of normal portland cement. This cement was usually combined with Ottawa sand and ground 0-No. 200 sieve in the proportion of 60 per cent by weight of cement and 40 per cent of silica. As pointed out in the introductory paragraphs, the presence of finely divided silica with the cement was found in the previous tests to contribute to the permanent strength and denseness of the hardening paste and to practically eliminate leaching and efflorescence. In these tests, therefore, 40 per cent of silica was used in most of the studies. As brought out later, in the lean mixes commonly used in block manufacture, the effect of the silica appears to be of less importance than in rich mixes used in some other types of concrete products.

The aggregates after oven-drying were separated into 5 sizes (0-No. 28, No. 28-14, No. 14-8, No. 8-4, and No. 4-⅜ in.) and recombined to give the grading desired. In most of the tests the 0-No. 28 material was replaced by silica sand graded No. 48-No. 28 in order to eliminate particles in the active size range (0-No. 48 sieve). In such tests the only silica present in the active size range was that in the cement-silica mixture.

Each batch was mixed 1 min. dry and 5 min. wet in a 3-cu. ft. Blystone paddle-type mixer. The amount of water added was determined by trial to produce a consistency as wet as practicable without impairing the handling properties of the units during and after molding.

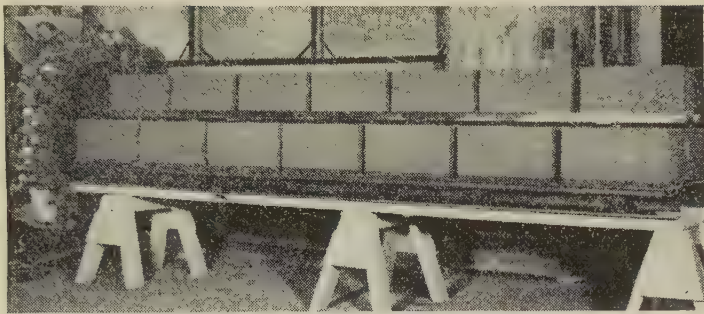

FIG. 2—FRESHLY-MOLDED BLOCK ON CAR READY FOR ROLLING INTO STEAMING CYLINDER

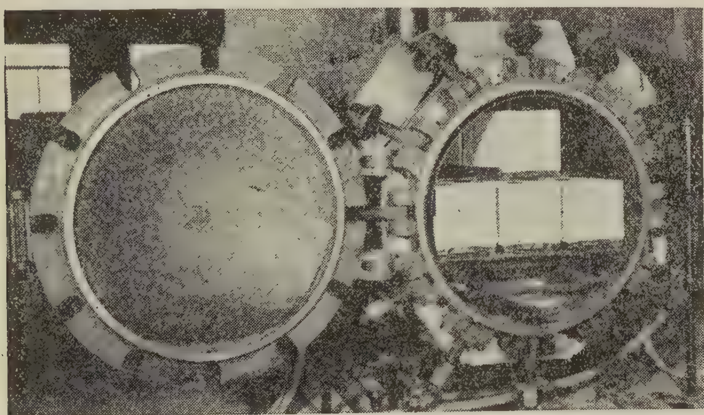

FIG. 3—STEAM-CURED BLOCK ON CAR READY FOR WITHDRAWAL FROM CYLINDER

Steaming of Block. Immediately after molding, the blocks were either placed in the steaming cylinder or were stored in the moist room at 70° F. until they were of the age desired for steaming. The block were placed on the car of the steaming cylinder as indicated by Fig. 2 and 3 so as to leave a clearance of about 1 in. between the sides and 1¼ in. between the ends of adjacent blocks. The top plate of the car on which the three lower rows of block were placed was perforated to facilitate the circulation of vapor and steam formed from the heated water in the bottom of the cylinder. Thermocouples indicated a negligible difference of temperature during the whole cycle between the water in the bottom of the cylinder, the vapor at the top of the cylinder, and the cylinder walls. The duplication of steaming runs was excellent.

Testing of Block. Usually the blocks were brought to constant weight in the air of the laboratory before testing for compressive strength, although some blocks were tested several hours after steaming and others were tested saturated with water. In general, duplicate steam-cured block samples are available for later tests.

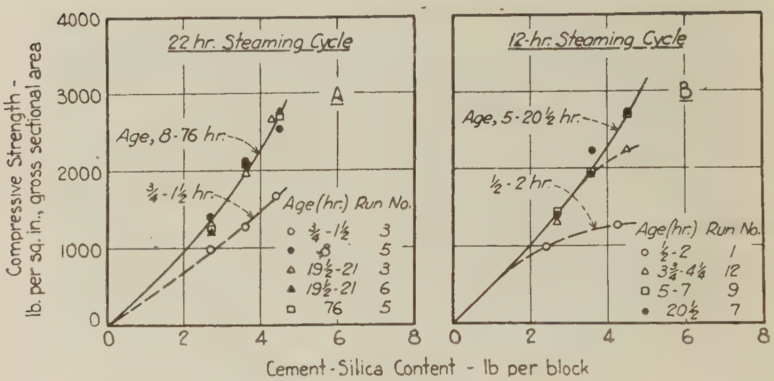

FIG. 4—EFFECT OF AGE OF BLOCK AT BEGINNING OF STEAMING PERIOD ON THE STRENGTH OF AIR-DRY BLOCK MADE WITH SAND AND GRAVEL GRADED NO. 48 TO ⅜ INCHES

DISCUSSION OF STRENGTH RESULTS

Tests with Sand and Gravel Block. Tests of block cured by the 22-hr. steaming cycle at ages of approximately 1, 8, 20 and 76 hr. (Fig. 4,*A*) indicate that:

1. When steaming was delayed until the age of 8 hr. the strengths obtained were from 35 to 65 per cent higher than that of blocks steamed at ¾ to 1½ hr. The improvement in strength increased with increase in the cement-silica content of the block.

2. No improvement or reduction in strength occurred by steaming at various ages between 8 and 76 hr.

3. The strength increased somewhat more rapidly than the increase in cementing material.

4. The strengths of steam-cured sand and gravel block 24 hr. after molding when compared with moist-cured block of similar mixes and characteristics (See JL. AM. CONCRETE INST., Nov., 1932, *Proc.*, v. 29, p. 120, Fig. 2) were substantially higher (varying from about 25 per cent higher for the leanest mixes to about 50 per cent higher for the richest mixes represented) than that of block cured in the moist room for 5 days at 70° F. and stored in air for 55 days before testing.

Tests of block cured in the short (12-hr.) steaming cycle at ages up to approximately 20 hr., (Fig. 4, *B*) indicate that:

1. Increased strength for the delayed steaming period as compared with steaming at ½ to 2 hr. after molding, similar to tests with the 22-hr. cycle. For the intermediate period—3¾ to 4¼ hr.—the leaner

mixes show strengths comparable to those of the longer curing period, but the richer mix gave an intermediate strength.

2. Compressive strengths obtained by steaming at the age of 5 hr. by the 12-hr. cycle equal to or more than that obtained in the 22-hr. steaming cycle by steaming at the age of 8 hr. to 3 days.

On the basis of the above it is concluded that satisfactory strength of sand and gravel units may be developed by following either the fast or slow steaming cycles used or modifications thereof provided that:

1. The period required for heating from 70-350° F. is not less than 3 hr.
2. The age when steamed is not less than 5 hr.
3. The exposure to saturated steam at 350° F. is not less than 8 hr.
4. The pressure release period is not less than ½ hr.

In general, it is believed that a better product will be obtained by a gradual cooling and pressure release period of at least 5 hr. or more. This is particularly desirable to avoid surface crazing in dense, rich concrete which is to be exposed to the weather.

Tests with Haydite Block. These results (Fig. 5, *A* and *B*) are similar in all respects to those with sand and gravel blocks. The effect of age at beginning of steaming is less marked and the increase in strength with increase in cement content is considerably less. The strengths of the steam-cured Haydite block were practically equivalent to those of good moist-cured Haydite block (See JL. AM. CONCRETE INST., Nov., 1932, *Proc.* v. 29, p. 120, Fig. 2).

Previous Results on Additions of Silica. In the earlier studies of high pressure steam curing[1] the presence of a certain amount of finely ground Ottawa silica (0-No. 200) with the cement showed a definite advantage with regard to strength, denseness of paste structure, leaching and efflorescence of steam-cured mortar specimens. These studies indicated a combination of the lime liberated by the hydration of the cement and the silica during the steaming treatment. Similar results were obtained with finely divided materials which were only partly siliceous such as Haydite, cinders, flue ash, lava, granite, etc. Usually, however, the percentages required for optimum strength with these materials were higher and the strengths lower than for mixtures of ground Ottawa silica sand and cement.

In view of the foregoing it is evident that when aggregates contain finely divided silica or silica-bearing material, less silica needs to be incorporated with the cement for optimum effect than when the

[1]"Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete," by Carl A. Menzel, Am. Concrete Inst., *Proceedings*, Vol. 31, p. 125.

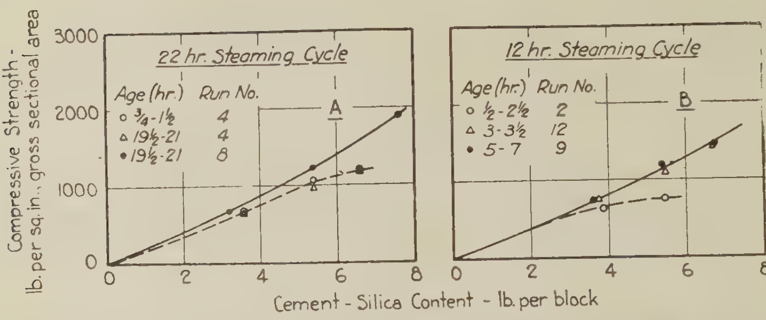

FIG. 5—EFFECT OF AGE OF BLOCK AT BEGINNING OF STEAMING PERIOD ON THE STRENGTH OF AIR-DRY BLOCK MADE WITH HAYDITE GRADED NO. 48 TO 3/8 INCHES

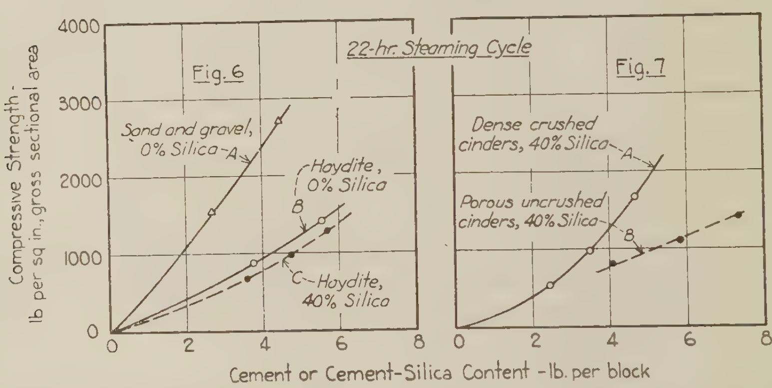

FIG. 6 AND 7—MISCELLANEOUS TESTS OF THE STRENGTH OF AIR-DRY BLOCK MADE WITH 0 TO 3/8 INCH AGGREGATE OF DIFFERENT TYPES WITH CEMENT OR CEMENT-SILICA MIXTURES

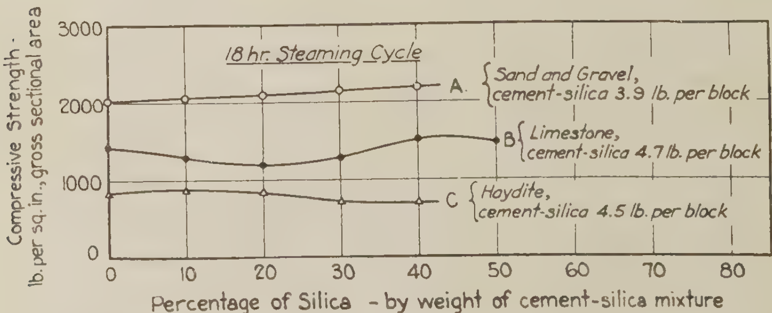

FIG. 8—EFFECT OF DIFFERENT PERCENTAGES OF FINE SILICA (0 TO NO. 200 SIEVE) ON STRENGTH. 0 TO 3/8-IN. AGGREGATE

aggregates contain no siliceous particles in the active size range. With lean mixes it is possible for some aggregates to provide all the finely divided siliceous particles required but with rich mixes there may be a substantial deficiency.

The early studies showed that with Haydite, the optimum percentage of 0-No. 200 material in the aggregate was about 30 per cent, the same as for pure silica, whereas for cinders it was about 60 per cent. These values indicate that in general for optimum effect more silica would have to be supplied for cinders than for Haydite of the same grading. The information available does not provide a basis for estimating the amount of silica it will be necessary to add for different types of aggregates as so much depends on the nature and amount of material in the 0-48 size range. The size distribution within this range is also important as the activity increases with the decrease in particle size.

Silica Additions to be Determined by Trial. In light of the results from the earlier tests just referred to it is evident that the products manufacturer will need to determine for himself whether silica additions are desired and if so the exact proportions to use with the particular aggregate, cement, and silica available. Such determinations can best be made by trial runs, using the materials in different proportions and testing the resulting units for strength. These trial runs should be made using mixes of the proportions of fine and coarse aggregate and the amount of cementing materials which will give the surface texture and strength desired.

In the various trials the proportion of silica to cement can be varied, but the total weight of the cementitious material to the aggregate should be kept constant. Three or four mixtures of fine siliceous material (0-No. 200 sieve) with cement should be tried ranging in silica content from 0 to about 40 per cent. The steaming cycle should be carried out in accordance with the recommendations given herein and the steam-cured units tested for compressive strength.

This procedure is illustrated by the results (Fig. 8) with blocks made with sand and gravel, limestone and Haydite aggregate which were steamed in an 18-hr. steaming cycle at the age of 5 hr. For a given aggregate the proportions by weight or the weight of cementing material (cement-silica mixture) per block were constant but the amount of silica in the mixture varied from 0 to 40 or 50 per cent. It will be apparent from a study of the individual strength curves that for the conditions of these tests (steaming cycle, cement, mix, etc.) approximately the same strengths were obtained without the admixture of fine silica as with different percentages of silica.

In view of these results it may appear that the use of silica additions is not justified and that the references to the earlier studies should not be cited as pertaining to block manufacture. However, there are certain factors not apparent from the strength data of the present series which should be given consideration. Cement-silica mixtures in optimum amounts continue to gain strength with time of exposure to steam at 350° F. Without silica or with low percentages of silica the development of strength may be erratic. With some cements the strength may increase rapidly and then decrease sharply with further exposure. With other cements the gain in strength may be quite slow even during exposure for periods of 2 or 3 days. The influence of silica in reducing efflorescence is also a factor. Therefore, until extended experience under actual plant conditions proves otherwise it appears desirable to determine whether or not silica should be used by trial somewhat as outlined to insure the best results with the materials available.

Supplementary Tests with Sand and Gravel and Haydite. Fig. 6 gives further evidence that good strengths can be obtained with some aggregates without the use of optimum silica-cement mixtures when the aggregate particles below the No. 48 sieve are siliceous or silica bearing.

The tests of sand and gravel and Haydite block (Fig. 6) were made with aggregate containing particles ranging from 0 to ⅜ in. instead of from No. 48 to ⅜ in. (Fig. 4 and 5). In other words, these tests were made with aggregate containing a normal amount of siliceous sand or silica-bearing Haydite particles in the active size range (0-No. 48 sieve) whereas in the other tests (Fig. 4 and 5) these were purposely omitted. Results when cement was used without the admixture of 0-No. 200 silica (*A* and *B* of Fig. 6) (the only fine silica present being the 0-No. 48 material introduced with the aggregate) are compared with results (*C*, Fig. 6) when the cement contained 40 per cent by weight of 0-No. 200 silica. In this latter case the total silica present was that introduced with the cement plus that provided by the 0-No. 48 portion of the aggregate. In contrast are the tests (Fig. 4 and 5) with only the 40 per cent of 0-No. 200 silica introduced with the cement.

Approximately the same strength was obtained with a given amount of cement alone per block with the 0 to ⅜-in. aggregate as with an equal amount of cement-silica mixture (60 per cent cement, 40 per cent 0-No. 200 silica) when used with No. 48 to ⅜-in. sand and gravel aggregate. (Compare Curve *A*, Fig. 6 with full-line curve, Fig. 4, *A*.)

Comparison (Curve *B* with *C* of Fig. 6) shows that with the same 0 to ⅜-in. Haydite aggregate somewhat higher strengths were obtained

with a given amount of cement alone than with the same quantity of cement-silica mixture. It appears possible that in one instance (Curve *B*) there was somewhat less silica and in the other (Curve *C*) somewhat more silica than the amount required by the cement for optimum strength. It is interesting to note that in the former tests (Curve *B*) somewhat higher strengths for a given amount of cement alone resulted than for the same amount of cement-silica mixture proportioned to contain approximately an optimum percentage of silica (*A*, Fig. 5). This is probably due to the presence of 0-No. 48 particles which served the double purpose of furnishing silica and building up a denser paste structure.

It may be well to point out in this connection that although the absence of 0-No. 48 particles constitutes a deficiency in grading which can be expected to result in lower strength and cement efficiency the gradings of the No. 48-3/8-in. and the 0-3/8-in. materials were not as different as they may appear. The two gradings were precisely the same for each size range above the No. 28 sieve and contained the same percentage of material below the No. 28 sieve except that in one case the material consisted entirely of particles ranging from No. 48-No. 28 size instead of particles ranging from 0-No. 28 size as in the other case. Although the 0-3/8 in. grading is preferable the strengths with the No. 48-3/8 in. material were only slightly lower and the absence of the fine 0-No. 48 material in most of these tests did not appear to constitute an important deficiency in the mechanical influence of grading on block strength.

Tests with Cinder Block. The block were made with two distinctly different types of cinders obtained by separating the same lot of power plant cinders ranging from dust to about 1-in. size on the 3/8-in. screen. One type, a light uncrushed porous cinders, comprised material passing the 3/8-in. screen. The other type consisted of material crushed from the 3/8 to 1-in. clinker particles and produced a relatively heavy, dense cinder aggregate with a unit weight about 50 per cent higher than for the porous, uncrushed cinders. Blocks made with the porous cinders contained only 23 lb. of aggregate compared with 35 lb. for the dense cinders.

The strength of block made with the dense crushed cinders and a cement-silica mixture containing 40 per cent of 0-No. 200 silica was substantially higher than that of the more porous, uncrushed cinders (Fig. 7). Even with the porous cinders, strengths can be obtained with cement-silica mixtures (*B*, Fig. 7) which are nearly equivalent to those shown for Haydite block (*A* and *B*, Fig. 5 and *B* and *C* Fig. 6). It may be expected from the results obtained separately with the two

types of cinders that a mixture of crushed and uncrushed cinders would give blocks intermediate in weight and having strengths substantially higher than indicated (Curve *B*) for the porous cinders alone.

Strength of Saturated Versus Air-Dry Block. Such data as are available indicate that the ratio of the compressive strength of steam-cured block in an air-dry condition to that of blocks in a saturated condition, is slightly higher than a similar ratio obtained in a large number of tests of moist-cured block. The strength of saturated steam-cured block averaged about 80 per cent for gravel and about 88 per cent for Haydite of that of similar block tested air-dry. For moist-cured blocks the corresponding values were about 75 per cent for gravel and 86 per cent for Haydite.

GENERAL COMMENTS ON VARIOUS FEATURES OF THE STUDY

Lightness of Color. One of the outstanding effects of the steaming treatment is to bleach the normally gray color of the cement to an almost white color. As a consequence all the steam-cured block were markedly lighter than similar moist-cured block.

Cracking. One of the important features of the tests was that no cracks occurred in any of the 8 x 8 x 16-in., 3-core block regardless of variations in the type of aggregate, mix, age at the start of steaming, or of the heating and cooling rate employed in the steaming cycle. All block gave a clear ringing sound when tapped with a hammer.

In this connection it is important to point out that during the heating period the transfer of heat from the vapor and steam to the concrete is accomplished without introducing large temperature differentials in the concrete. This is because the saturated vapor and steam is brought into contact with all exterior and interior (core hole) surfaces of the hollow units. During the early part of the cooling period the temperature of the concrete tends to be equalized by the heat absorbed in converting some of the free moisture in the concrete from liquid to vapor as the vapor pressure of the surrounding steam is decreased. This action is enhanced by the hollow structure of the masonry unit and by the porous nature of the concrete. The latter part of the cooling period, when the units are withdrawn from the cylinder and exposed to the air, probably represents the most critical period from the standpoint of cracking and surface checking. At this stage, where a good portion of the free water has been removed from the concrete, further loss of moisture is accompanied by an increasing rate of shrinkage. Hence, the concrete has to withstand stresses set up both by moisture and temperature differentials. Although there appeared to be no cracking in either the face shells or webs, the rough

surface texture of the units made it impossible to determine with certainty whether or not the surface was crazed with very fine checks or cracks.

Moisture Loss During Steaming. All steam-cured concrete had a lower moisture content after steaming than before. It appears that little, if any, moisture loss could occur during the heating period, as the concrete is in an atmosphere of saturated water vapor which is constantly condensing on the cooler concrete surfaces. Thus, heating of the concrete may actually be accompanied by an absorption rather than a loss of moisture. During the constant temperature and pressure period there is very little interchange of heat between the steam and the concrete, consequently the specimens will neither gain nor lose moisture. During the cooling period, however, the temperature of the concrete and of its contained moisture is higher than that of the surrounding steam or water vapor. Consequently the vapor pressure of the moisture in the concrete is higher than that of the steam, and with the tendency to attain equilibrium, moisture is lost by the specimen. Upon removal from the cylinder there is still a difference between the vapor pressure of the moisture in the warm concrete and that of the vapor of the surrounding air and more moisture is lost until approximate equilibrium is attained.

The tests showed that of the total moisture lost by the block from the freshly-molded to the ultimate room-dry condition after curing, about $\frac{2}{3}$ in the case of the Haydite and porous cinder block, and $\frac{9}{10}$ in the case of the gravel and dense cinder block, was lost during the period in the cylinder. Large differences in the steaming cycle, cooling and pressure release periods did not appear to influence the extent of the moisture loss in the cylinder.

Brittleness. Steam-cured blocks made with sand and gravel aggregate appeared to be as resistant to damage in handling as moist-cured blocks. They usually had sharp, strong edges and corners and the top surfaces did not show any excessive tendency to crumble. Steam-cured Haydite block, however, appeared to be somewhat more brittle and susceptible to crumbling at the corners, edges and top surfaces than good moist-cured block. This susceptibility to damage was even more pronounced with steam-cured cinder block.

Lubrication of Pallets. The lubrication of the pallets with oil or grease was very unsatisfactory. These lubricants were absorbed by the hot concrete, discolored it, and failed to prevent bonding between the concrete and the pallets during steaming. A mixture of 1 part by weight of stearic acid and 2½ parts of kerosene used by some manu-

factures of cast stone proved to be very satisfactory. This mixture had none of the disadvantages of the grease or oil.

Precaution Against Discoloration. Possible discoloration of the block was prevented largely by the dripping of condensed water by a thin curved sheet of steel placed about ½ in. from the upper half of the cylinder to deflect the condensation. The malleable iron pallets did not appear to rust or discolor the block nor did dripping from the upper course of blocks appear to discolor those in the next lower course. The cumulative effect of the condensation from a series of vertical courses may, however, present a problem in the steam curing of high quality products in cylinder of commercial size, 6 or 8 ft. in diameter.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for March-April, 1936. Discussion should reach the Secretary by February 1, 1936.*

Placing Concrete by Means of Vibration

Introducing the Work of A. C. I. Committee 609, Vibration of Concrete

ALFRED E. LINDAU, CHAIRMAN

Editor's Note—*Three papers presented here, by* Ben Moreell, Sam Comess *and* T. C. Powers *conclude the work presented by* Mr. Lindau's *Committee at the 31st Annual Convention, New York, February 19-21, 1935. Introductory remarks by the Committee's Chairman and papers by* C. M. Hathaway, F. V. Reagel *and* W. R. Johnson *were published in the March-April* Journal. *In the May-June* Journal (*Proceedings Vol. 31*) *the series was continued with papers by* M. O. Withey, Thomas E. Stanton, Jr., Lewis H. Tuthill *and* F. H. Jackson. *Discussion of the entire series is now open—closing February 1, 1936, with a view to publication in the March-April* Journal. *Meanwhile Committee 609 is working on a report involving tentative recommendations of practice in the use of vibration as a means of placing concrete.*

Concrete Vibrating Practices in France

BY B. MOREELL*
MEMBER AMERICAN CONCRETE INSTITUTE

I have been asked to say something about concrete vibrating equipment and practices in France.

In general, the equipment now available is similar to the equipment in this country, with one exception. I refer to what they call "floating per-vibrators." "Per-vibration" is a trade-marked term, the property of La Societe des Procédes Technique de Construction, a French company. The term "Per-vibration" is meant to denote "internal vibration" by means of equipment manufactured by that company. The "floating vibrator" consists essentially of a heavy metal shell, inside of which is a vibrating element. The shell has a cone or wedge shape so that when the concrete surrounding the shell is reduced to a quasi-liquid state by the vibratory action, the bouyant force of the displaced concrete causes the vibrator to float to the surface. The vibrator is placed in the bottom of the form, concrete is dumped into the form, and the vibrating element is then started. The vibrator assumes a vertical position and works its way to the surface. The optimum rate of rise has been estimated to be 13 feet per hour.

These "per-vibrators" are made in several types. One is designed for concrete piles, poles, and mass concrete; another especially for walls; a third is the familiar type of hand vibrator; still another, for floor slabs, consists of a pan of heavy sheet metal on which is mounted the vibrating element. I witnessed the operation of this vibrator on the 4-in. floor slab of a hospital building, and it seemed to be doing its work very satisfactorily, as did the one used for mass concrete.

These vibrators are operated by compressed air. In 1932 and the early part of 1933, all of the vibrating equipment which I saw in operation in France was operated by air, but the company producing this equipment was then experimenting with electric drive. The equipment which I saw was not as rugged as our equipment and not as fast; the air motors delivered only 1800 blows per minute. I am

*Lieut.-Commander (when this paper was presented—now Commander) Bureau of Yards and Docks, Navy Dept., Washington, D. C.

informed that since that time suitable electric vibrators have been produced and are now in satisfactory operation.

Internal vibration is greatly favored over vibration of forms, but where the former is impracticable vibrating of the forms is practiced. I visited a job at Joinville, just outside of Paris where pneumatic vibrators were rigidly attached to the form of a sewer tunnel. They were boring through soft rock and placing a lining of slag cement concrete about 3 in. thick. The vibrator was apparently very effective.

A platform vibrator is similar to the Jackson platform vibrator. A variation of this type is made by attaching metal spuds to the under side of the platform. The manufacturer claims that this apparatus has the advantages of both surface and internal vibrators.

A somewhat unusual type of vibrator, designed for placing large areas of mass concrete, consists essentially of a platform of expanded metal, on top of which is mounted a frame-work of steel pipe which can be weighted, as required. Mounted on this frame-work is a pneumatic vibrating element, or, in the larger sizes, several such elements. The expanded metal rests on the surface of the concrete and it is claimed that a very effective vibrating action is obtained by virtue of the interlocking of the elements of the expanded metal and the large aggregate particles. Another advantage claimed for this apparatus is that the surface is left rough and, therefore, in ideal condition for bonding to the subsequent layers of concrete. The weight and size of the apparatus can be varied to meet the needs of the job. Such apparatus has been made with a platform as large as 3 ft. 6 in. x 13 ft. with eight vibrating elements.

The use of vibratory equipment was becoming popular in France when I left there in the summer of 1933, but the economy of compacting by vibration in lieu of hand-tamping is not as marked in France as in this country because of their lower wage scale.

It should be noted, also, that the Allied Machinery Co. is distributing the vibrators of the Electric Tamper & Equipment Co., but it is difficult for this American company to compete, as most of the large work in France is under the control of the Government and equipment of French manufacture is favored.

While some experimental studies of vibration have been made in France these have not been as extensive nor as numerous as American investigations. The conclusions reached from the investigations made so far are in close agreement with those arrived at in this country.

(*See Editor's Note, page 65*)

Practical Applications of Vibration For Placing Concrete*

BY SAM COMESS†

Vibration was first used in the Rock Island district in 1932, when Dam No. 15, Davenport Seawall and Sewer, on the Mississippi River, was in process of construction. Here the specifications required a six-sack mix, with maximum size of coarse aggregate 1½ in. Crushed stone was used in the concrete for the dam and seawall. The sewer was built of gravel concrete. The use of vibrators was optional and was required only where, due to the conditions of the job, more satisfactory results could be obtained than by hand placement. Here the concrete in the sewer sections and in the inaccessible parts of the piers and abutments of the dam was placed with the aid of vibrators. The machines used were the electrically driven, flexible shaft, "sausage" type. This flexible shaft vibrator worked especially well in highly reinforced sections, thin walls and other restricted areas.

To avoid further difficulty, if possible, with excessive temperature cracks, largely due to volume changes, specifications on future work were modified to provide a low-slump, low-cement content concrete to be placed by vibrators.

In the last year, concrete construction has been practically completed on seven locks and one dam. Of this total of eight projects, six used crushed stone as coarse aggregate and two used gravel. In a brief, general way, experiences in the use of vibrators in placing concrete in this district are described herein.

The manner of vibrating treatment varies with the placing condition and the type of concrete. It has been learned that no definite time of vibration, size of batch handled or thickness of horizontal layers placed can be set. It has also been learned that each concrete, due to its physical characteristics, requires a different vibration treatment, this being determined on the job under actual placing conditions.

*Observations cited herein were of Mississippi River lock and dam projects, Rock Island District, under direction of the District Engineer, Lt. Col. R. A. Wheeler, Corps of Engineers.—Author.

†District Concrete Technician, Rock Island District, U. S. Corps of Engineers.

As an illustration, let me compare two typical concretes used on the different jobs and manner of treatment that was necessary to get satisfactory results. In one of the concretes, due to the appreciable quantity of water gain or "bleeding" of free water, it was necessary first to maintain the total water of the mix at a minimum. Consequently, the mix was rendered harsh, and non-plastic. Secondly, to combat this excessive water gain and its probable effect on finished surfaces, the mass was placed in comparatively shallow layers of 8 in. to 12 in. and vibrated thoroughly by large vibrators. Following this treatment, smaller vibrators were employed, working very close to the form face without touching it. Free water after this treatment was taken care of by bailing.

Compared to the above mentioned concrete is another crushed stone concrete of the same unit paste content, but more plastic, cohesive and workable. This concrete was placed in horizontal layers of 18 in. or more and molded in place by large vibrators. The length of time of vibration was not as great as that for the first described mix. This concrete was more easily molded into place and followed by light treatment with the smaller vibrator, satisfactory placing results were obtained. Reference to Table 1 offers comparative data on the different types of vibratory treatment used.

On two jobs a gravel concrete with a maximum of 1½-in. coarse aggregate was used. This gradation of coarse aggregates made it necessary to use an additional quarter sack of cement. Although different methods were employed in transporting the concrete on the two jobs, approximately the same vibration procedure was followed. The concrete was placed in approximately 12-in. layers thoroughly vibrated with the large (2-man) vibrators, supplemented by spading, giving satisfactory results.

TABLE 1—COMPARATIVE RESULTS OF TREATMENT REQUIRED BY THE DIFFERENT CONCRETES FOR SATISFACTORY RESULTS

No.	Coarse Aggregate	Slump	Vibration Treatment
Type 1	White crushed limestone; 2-in. maximum aggregate	¾—1¼ in.	Vibration with large vibrators followed by spades, and treated with small vibrator working close to the face.
Type 2	Gravel; 1½ in. maximum aggregate	¾—1 in.	Vibration with large vibrators followed by spading.
Type 3	Brown crystalline limestone of high absorption; 2-in. maximum aggregate	¾ in.	Vibration with large vibrators followed with vibration close to the face with lighter vibrators.

Note—The cement content varies between the limits of 4.5 sacks –4.75 sacks per cubic yard. The frequency of vibration was rated at 3600 per minute.

Bleeding of water out of the mass during placing was especially bad on several of our jobs but this condition was due mainly to the physical

properties of the mix. When the mix was of this nature, puddles of water would form in the pockets made by the vibrator. Whenever this water gain condition was prevalent, the concrete crews would have a tendency to under-vibrate for fear of stirring up the mass too much. Hence, great care was exercised in attempting to keep the water gain at a minimum by controlling the mix and not affect the quantity of vibration necessary for proper placement. It is probable that vibration caused the "bleeding" to make its appearance at a more rapid rate. Whenever the mix was of a cohesive, plastic appearance, the vibrator had no tendency to cause "bleeding" or water gain from the mass, within the length of the vibrating period used.

As far as could be observed the so-called lighter vibrators (1-man) proved to be ineffective in handling the mass in the necessary time to avoid the piling up of concrete in the form. In every case, with the consistency of the mix used, the heavier and more rugged type of vibrator was necessary for thorough compaction of the mass. Any segregation was reconsolidated. In bucket placement, good vibration results were obtained by starting at the point of deposit with two heavy vibrators held in an inclined position and pulled forward over the batch. This operation handled the batch in sufficient time for continuous placing. In the case of the pumpcrete method of placement, one heavy vibrator operating at the point fo deposit was found to be adequate for the settling of the mass. However, this was followed by further vibration treatment as found to be necessary.

In every case, the lighter vibrators (1-man) or the flexible shaft type was used for the lighter work—such as vibrating close to the face, difficult placement around recesses—forms in which there was steel, or in conjunction with spading, or in places inaccessible to the heavier type of vibrator.

To attempt to determine length of vibration, effective distance of the vibrator from the form face and benefits derived by supplementing vibration with spading, a job study was undertaken, using a form, part of one side of which was built of glass. From this study, of this particular crushed stone concrete (1½-in. slump) it was found that 15-20 seconds was adequate vibratory treatment in placing the mass, that the center of the vibrator should not be held further than 6 in. from the form face for effective surface treatment and that there was some benefit derived by supplementing vibration with spading. The size of form was 7 x 5 x 3 ft.; concrete placed by pump in ½ cu. yd. layers. The mix was 1:3.06:4.77 by weight, w/c .87 with 4.75 sacks per cu. yd.

In applying this practically on the job, vibration was judged completed when the quantity of air bubbles emanating from the mass was reduced to a minimum—as can be judged by the naked eye. It was found that by properly training the vibrator men in the form, that the treatment and length of vibration accorded each batch or pile of concrete in the form was of approximately the same kind and duration. What separation of mortar from the coarse aggregate there was, was distributed with shovels and vibrated back into the mass.

As learned from this test, the vibrator is most effective in the dispersion and liberating of air bubbles from the face when held as closely as possible to the form without touching it. This was substantiated on the job, when at the early stages of the concreting operations, the vibrators were held 12 in. or more away from the form face. Upon stripping of the forms, large air bubbles literally covered the surface. These bubbles were deep and elongated, and proved to be typical as to shape and size, of that obtained in the test. By vibration closer to the face, the bubbles apparently were dispersed or to a large extent liberated, for in future forms the size of the bubbles was appreciably reduced. In both cases, vibration was supplemented by spading.

The benefits derived by spading after vibration depends largely on the consistency and plasticity of the concrete to be placed. Whenever the concrete is of stiff but plastic consistency, one that will work up readily by spading in conjunction with light vibration, there appears to be a decrease in the number and size of the air bubbles on the surface. In a concrete that is harsh-working, non-plastic, it is believed that no benefit is obtained by spading. In this case, it is believed that vigorous, systemized vibration close to the form face will give satisfactory results. At any rate, this proved to be the case in two instances, when the concrete was of a harsh, non-plastic nature.

In my opinion, the number and severity of the air bubble condition on surfaces are increased as a result of the "dry," consistency, lean mix, non-plastic concrete placed under vibration. However, wherever the mix was sufficiently plastic to allow thorough spading, the surfaces proved to be comparable to those placed by the hand-placing method.

As a means of determining the effectiveness of vibration on concrete of the lean-mix, low-heat cement specification, concrete cores were drilled from each job. Each core was studied for distribution of aggregate, number and size of air pockets. Tests on compression, abrasion, absorption and freezing and thawing were also included in this study. It was interesting to note that when the mix was harsh,

non-plastic, and stiff (¾ in. slump) as compared to another mix which was plastic, workable, and of approximately the same slump, there were a greater number of air pockets present. In both cases the specimens showed an even and homogeneous distribution of coarse aggregates. It appears that the influence of the same size of vibrator in these two mixes is not the same for equal radii.

CONCLUSIONS

From observations and experiences with a concrete produced under a specification radically different from a "bootable" hand placed concrete the following general conclusions are drawn.

1. Each concrete produced under different local conditions requires some change in vibration treatments. The manner of this treatment depends largely upon the physical characteristics of the mix and method of placement.

2. For concrete, using minimum cement content of 4.5 and 2-in. maximum aggregate, the heavier, more rugged type of vibrator is necessary. The larger units generally used on the jobs in this district weigh about 75 lb. and are handled by two men.

3. The most effective "melting" of the batch is produced by placing the vibrator on the side of the batch near the base of the cone formed in placing. After the cone formed by the batch has been reduced the remainder of the batch is treated.

4. Concrete that is cohesive and plastic in the form, where vibrating is supplemented by spading, tends to minimize the air bubbles on the surface. It was learned that vibrating several feet in advance of the spading, followed by light vibration after spading gave satisfactory results.

5. To avoid any chance of getting a concrete which gives off appreciable free water, a more rigid choice of concrete materials should be made. Water gain, or "bleeding" of free water, is a result of the physical characteristics of one or more of the individual materials of the mix and not a result of vibration. Vibration may produce this condition sooner.

6. Vibrators should not be allowed to touch the form faces. The surfaces of the concrete have been considerably roughened by this practice and it has been detrimental to the life of the form. It has been learned that keeping the center of the vibrator approximately 6 in. from the face will give effective results.

7. The consistency of the concrete as measured by the slump cone should not be less than ¾ in. A slump less than ¾ in. did not react very readily under the influence of the vibrators used. A field measurement as to the minimum slump was determined as the point in which the hole formed by the vibrator closed after the vibrator was withdrawn. Concrete which still retained the shape of the vibrator spud after being withdrawn was considered too stiff to be placed properly with the vibrators used. The range of slumps varied from ¾ in. to 2 in. depending on the type of concrete and job conditions.

8. The length of time of vibration and vibrating treatment must be governed by the job. It is probable that vibration of higher frequency than 3600 per minute may be necessary for placing no-slump concrete, but whether it is practical, from the standpoint of job maintenance or economy, would require study on the individual job.

(*See Editor's Note, page 65*)

Observations on the Use of Vibration in the Field

BY T. C. POWERS*
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During 1934, following the laboratory studies previously reported,† the writer was privileged to observe the use of vibrators on several projects. On some, their use was eminently successful; on others there was much room for improvement. Whenever unsatisfactory results were being obtained, the causes were usually found among the following:

1. Poor management of the placing crew.
2. Improper methods of transporting concrete from the mixer to proper place in the forms.
3. Unwise selection of size or number of vibrators.
4. Vibrators operating below normal speed.
5. Improperly designed mixes.

MANAGEMENT AND METHODS

The first two items are, of course, the common source of poor results when concreting by any method. There is not much to be said about these, for no amount of exhortation will produce foresight, good judgment, attention to details, and the ability to coordinate various operations. When concrete is carried to the forms and deposited in such a manner that the whole operation degenerates to a helter-skelter game of tag between the concrete bucket and the placing crew, or when the vibrator men wander like aimless nomads from hillock to hillock of concrete, there is only one remedy—change the management.

The thoughtful superintendent will not need to be reminded that the placing crew must work systematically, covering the whole area without a skip here and an over-vibration there; that the concrete must be deposited in such way as to make systematic work possible, and that each individual of the crew must work to produce a certain final appearance, rather than to "stay put" until some distant hummock lures him away, or until he receives orders to move.

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†Vibrated Concrete, by T. C. Powers, Journal, Am. Concrete Inst., June 1933, *Proceedings*, V. 29, p. 373.

Neither will the wise superintendent try to make one vibrator do the work of two, nor use a ¾ h. p. machine where three times this power is necessary. But in this latter respect errors are excusable for it is believed that few, if any, (certainly not the writer) have had enough experience with vibrators to be able always to predict the number and size of machines required for a given job.

SIZE AND NUMBER OF VIBRATORS

Once a job is under way, it should be possible to recognize a deficiency in either number or size of machines, although to do so it may be necessary first to investigate the suitability of the mix. This latter step is required because an efficient machine may be ineffective due to a mix too far outside the range suitable to the machine, i. e., either too plastic or too dry and harsh. In fact, it is the writer's belief that until a better, more direct method can be devised, the efficiency of a machine can best be judged in terms of the type of mix that it can handle.

On this basis, efficiency may be considered as being proportional to the amount by which the water content can be lowered below that of a mix suitable for placing by hand. Since the water content may be lowered either by using a drier consistency or a coarser gradation, the process of judging the vibrator becomes one of designing a mix in which gradation and consistency are combined in a way most suitable for the vibrator.

DESIGNING MIXES

To find mixes best suited to a given set of conditions the writer has employed successfully a method of designing in the field, using full size batches and the regular mixing and placing crew and equipment. The method follows the same general approach as that reported in the paper "Studies of Workability of Concrete."* The "rule of constant water content" (to be given later) is employed in a systematic series of trial mixes. When working under specifications that require weighing equipment and other refinements of control, the results are more reliable than those obtained from laboratory tests, if proper precautions are taken. Indeed, it is almost impossible to design mixes for vibrators in the laboratory, for there is no way of telling (now, at least) how the size of the mass will affect the efficiency of the vibrator.

To simplify the description, let it be assumed that only one gradation of sand and one of coarse aggregate is involved. Also, assume that the number of vibrators to use has been tentatively decided. The problem thus becomes one of choosing a proportion of fine to coarse,

*Studies of Workability of Concrete, by T. C. Powers; JOURNAL AMERICAN CONCRETE INSTITUTE Feb., 1932 (*Proc.* V. 28, p. 419)

and a cement and water content which will meet the specifications and at the same time work the vibrators to capacity.

The first step is to choose a mix, either on the basis of previous experience, or preliminary laboratory tests. The mix should be such as to require a water ratio slightly below that specified, and such as to appear somewhat oversanded; that is, one that has greater plasticity than is required by placing conditions. With this mix, the regular placing operation is started using the minimum amount of water which will permit placing at the required rate. It should be kept in mind that when working with full size equipment in the field, it is not feasible to maintain a constant water-cement ratio during a trial run. It is far more practicable to use water in the quantities necessary for the vibrators to handle the concrete at the desired rate.

There should be available at this time accurate data as to the water carried into the batch on the surface of the aggregate or water absorbed from the batch if the aggregates are dry. With the above information, together with the amount of water added, and the weights of the other ingredients, the total quantity of water per cu. yd. of concrete is computed.

The next step is a slight reduction in the percentage of sand with, of course, a compensating increase in the percentage of gravel. Adjustments of consistency are again made by varying the added water until the minimum amount of water which will permit the vibrators to keep up with production is determined. It will be found that as the percentage of sand is reduced, the total quantity of water per cu. yd. may likewise be reduced. The quantity of cement in the batch is allowed to remain unchanged.

This procedure is continued with successive small reductions in the percentage of sand until a point is reached where no further reductions in the water content are made possible by reductions in sand. At this point the mix will appear too harsh. To overcome the effect of this harshness and to permit placing the concrete at the required rate, extra water, i. e., a wetter consistency will be required. This need for an increase in the water is definite evidence that less than the proper quantity of sand is being used. Having determined this absolute minimum in sand percentage, the next step is to hold the total water constant and increase the percentage of sand up to the maximum which can be used with this minimum quantity of water.

At this point, if the composition of the mix is checked, it will probably be found that the water-cement ratio is not the same as that specified. Compensation for this is made extremely simple by what may be

called the rule of constant water content as stated by Prof. Inge Lyse.* It has been found that in a given series of mixes of the same consistency and using the same aggregate combination, the total amount of water per cu. yd. of concrete remains substantially constant regardless of the variation in cement content. This general statement will not apply to changes in cement content over the full range of practical mixes, but it will apply very closely within the range involved in the process under discussion.

During the above operation the technician should make the necessary slide rule calculations of the composition of the several trial mixes. By these he will know at this time the minimum quantity of water which can be used—let us say that it is 30 gal. per cu. yd. If the specified water-cement ratio is 6.0 gal. per sack, he will know also that the cement content must be 30 gal. per cu. yd. divided by 6 gal. per sack or 5.0 sacks of cement per cu. yd. If the actual cement content in the final mix of the trial series is other than 5, the water-cement ratio is wrong and hence the cement content must be changed so as to give the correct value.

To illustrate this correction, suppose that the actual cement content was found to be 5.5 sacks per cu. yd., instead of 5.0; this would give a water-cement ratio of 5.45 gal. per sack. Obviously, the cement content must be reduced .50 sacks per cu. yd. to obtain full advantage of the specified water-cement ratio. It is necessary to compensate for the decrease in cement content by an increase of an equal absolute volume in one of the other ingredients, in order to preserve constant consistency. Since the water content must remain constant, the increase must be made in either the fine or coarse aggregate. Knowing that the percentage of sand should usually increase with a decrease in cement content, it is logical to make the compensating change in volume by increasing the quantity of sand.

This completes the process of designing when only two sizes of aggregates are involved. It is important to note that at no point in this process is it necessary to interrupt or slow down the regular process of placing. It is necessary, however, to take certain precautions if good results are to be obtained. For example, it is almost impossible to design mixes by this method if the moisture contents of the aggregates are subject to frequent variations. If the consistency changes continually from batch to batch because of variations in the moisture content in the sand, it is extremely difficult to determine either the amount of added water required for the necessary consistency or the total amount of water included in the batch. Difficulties

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of this kind can usually be circumvented by an investigation and correction of the source of the consistency variation.

During the trial run the technician should be posted at the forms where he can observe all of the successive mixes. He must be in close communication with the mixing plant, however, either by means of telephone or messenger so that changes in water or proportions can be made according to his instructions without delay.

It is considered outside the scope of this paper to discuss the matter of the individual gradations of the fine and coarse aggregates. As far as the coarse aggregate is concerned it is enough to say here that by varying its grading in a systematic manner, and treating each variation just as described above, the best gradation or gradations, or the maximum or minimum amount of some particular size that can be economically used, can easily be determined.

The gradation of sand is usually less subject to manipulation. But often, even with what appears to be the optimum percentage, excessive "bleeding" of free water occurs during or following vibration. In this case it is necessary to supply additional material finer than the No. 48 and 100 sieves.

INTERPRETING RESULTS

The process described should result in a mix which can with confidence be defended as the most suitable one that can be produced from the given materials. If it is found that there is no substantial difference between this mix and one suitable for hand placing, either there are too few vibrators or the vibrators are too small and underpowered. The chances are that the observer will have decided which is the case before the trials are finished. If, for example, the vibrators do not visibly affect the concrete over a radius of at least 18 in. around the point of application, or if one or more vibrators will not set most or all of a batch into a state of lively vibration, the vibrators are too small.

Two underpowered vibrators will not do the work of one that is adequately powered. It appears that some minimum impulse is necessary to set a given mass in vibration, and that impulses less than this minimum are only partially additive. If vibrators are obviously effective but are needed in two or more places at the same time, there are not enough of them.

In either case, it is necessary to stop short of the full possibilities of the type of vibrator employed. To employ more, sufficiently large machines will, under these conditions, permit the use of less water, either through use of a drier consistency, less sand, or both. How far to go in this direction will, of course, depend on circumstances peculiar

to the job, but if the problem is approached in a systematic manner such as is described above, final decision should not be difficult.

This paper obviously contributes nothing fundamental toward the solution of the problem of successfully applying vibration to concrete. At best it can only suggest a purely empirical or practical approach. Until the mechanics of vibration are quantitatively solved in terms of the properties of the mix, there seems to be no other basis upon which to work.

In reference to discussion of this and two preceding papers see editor's note, page 65.

Properties of Mortars and Concretes Containing Portland-Puzzolan Cements*

BY RAYMOND E. DAVIS, J. W. KELLY, G. E. TROXELL, AND HARMER E. DAVIS†

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SUMMARY

This paper presents results of a cooperative investigation of portland-puzzolan cements to determine the effect of chemical composition, physical character, and proportion of puzzolan upon the strength, volume change, resistance to the action of sodium sulfate, and other properties of mortars and concretes.

Comparing portland-puzzolan cements as a group with portland cements as a group, the former are more grindable; produce more plastic concretes exhibiting less water gain; generate less heat of hydration; and produce more impermeable concrete; but require more water to produce mortars and concretes of a given consistency, and exhibit greater shrinkage of mortar upon drying.

It has been found that there is a fair correlation between the activity of a puzzolan (as determined by the compressive strength of puzzolan-lime-sand mortar) and the compressive strength at the later ages of mortar containing the corresponding portland-puzzolan cement. In general, for rich mixes the compressive strength of concrete is less for portland-puzzolan cements than for the corresponding portland cements; but for the leaner mixes and active puzzolans the compressive strength of concrete at the later ages is greater for portland-puzzolan cements than for the corresponding portland cements.

Portland-puzzolan cements containing the active puzzolans high in silica exhibit materially greater resistance to the action of sodium sulfate than the corresponding portland cements; and the larger the percentage of puzzolan the greater the resistance.

In all of the cases investigated, the calcination of a puzzolan reduces the shrinkage upon drying of mortar containing the corresponding

*Progress report presented at the 31st Annual Convention, American Concrete Institute, New York, February 19-21, 1935. As here revised to include 1-year tests, received by the Institute Secretary, September 19, 1935.

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portland-puzzolan cement. In some cases, calcination of the puzzolan increases the strength and sulfate resistance for the corresponding portland-puzzolan cement, but in other cases the reverse is true.

Considering the various groups of natural puzzolans, those of diatomaceous origin are the most active, followed in order by the volcanic silicas, clays, and siliceous rocks.

Limestone dust used as a blending material in a portland cement increases the resistance to sulfate action and produces mortars and concretes of moderately high strength. Portland-puzzolan cements containing limestone dust exhibit lower shrinkage of mortar in air than do any of the portland-puzzolan cements except the one containing pulverized Ottawa sand.

Considering the various properties of concretes investigated, it appears that the optimum amount of puzzolan which may be employed in the manufacture of a portland-puzzolan cement may lie between 10 and 30 per cent, depending upon the type of puzzolan, the composition of the portland-cement clinker, and the conditions under which the cement is to be used.

Of the four types of portland-cement clinker investigated, portland-puzzolan cements manufactured from the clinker of high lime content appear to give most favorable results so far as strength and volume-constancy are concerned.

These investigations make it appear that portland-puzzolan cements of proper composition are suitable for hydraulic structures, mass concrete, and structures subjected to the action of aggressive waters. They are suitable for general concrete construction where extreme drying conditions are not encountered; but, because of their relatively high shrinkage, portland-puzzolan cements containing a large percentage of puzzolan are unsuitable for rich mixes in thin sections subjected to prolonged drying.

As compared with the corresponding portland cements, the relatively high workability, impermeability, and strength of portland-puzzolan cements *in the leaner mixes* make these cements particularly suitable for concretes in which strength is not a primary requirement. In such cases, there appears the possible economic advantage of using less portland-puzzolan cement than would be required to secure the same results if a portland cement were employed.

INTRODUCTION

The general term "puzzolan" applies strictly to a siliceous material which in itself has no cementing value but which exhibits cementitious properties when mixed with hydrated lime. In this paper, however,

the term is used to designate all materials investigated for puzzolanic activity, whether they be active or inert. The portland-puzzolan cements described herein were manufactured by intergrinding portland cements and puzzolans. For definitions, historical information, and a brief bibliography relating to high-silica cements and puzzolans in general, reference is made to a previous Institute paper.[1]

Scope and Purpose

Tests were made on 18 types of puzzolan (including the inert materials) and 4 types of portland-cement clinker, comprising in all 88 cements. The effects of the following factors were investigated:

a. Chemical composition of portland-cement clinker
b. Chemical composition of puzzolan
c. Physical character of puzzolan (solid-grained *vs.* diatomaceous, etc.)
d. Calcination of puzzolan
e. Proportion of puzzolan in cement
f. Fineness of portland-puzzolan cement
g. Specific gravity
h. Age at test

The foregoing factors were investigated in relation to their effect upon the following properties:

A. Grindability of cement
B. Water requirement for fixed consistency of neat paste, mortar, and concrete
C. Time of setting of cement
D. Soundness of cement
E. Tensile and compressive strength of mortar
F. Compressive strength of concrete
G. Elasticity of concrete
H. Unit weight of concrete
I. Absorption of concrete
J. Heat of hydration of cement; strength-heat ratio
K. Volume changes of mortar
L. Resistance of concrete to action of freezing and thawing
M. Resistance of neat paste, mortar, and concrete to action of sulfate waters

The program was designed to develop information relative to (1) a reliable method of test to determine the suitability of various types of puzzolan, (2) the optimum percentage of puzzolan, (3) the optimum chemical composition or type of portland-cement clinker, and (4) the optimum degree of fineness, considering costs in relation to the properties of the concretes. The aim of the investigation was to provide information which might be used as a guide in the preparation of specifications for cements of this type and for the use of such cements in concrete construction.

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[1]Properties of Mortars and Concretes Containing High-silica Cements, by Raymond E. Davis, R. W. Carlson, J. W. Kelly, and G. E. Troxell, JOURNAL, Amer. Concrete Inst., Vol. 5, March-April 1934 (*Proceedings*, Vol. 30), pp. 369-388.

Cooperating Agencies

The tests were conducted in the following laboratories:

California Portland Cement Co., W. C. Hanna, Chief Chemist.

Engineering Materials Laboratory, University of California, Raymond E. Davis, Director.

Metropolitan Water District of Southern California, Lewis H. Tuthill, Testing Engineer.

Santa Cruz Portland Cement Co., R. A. Kinzie, Jr., Chemist.

Southwestern Portland Cement Co., R. M. Willson, Chief Chemist.

Grateful acknowledgment is made to Messrs. Hanna, Tuthill, Kinzie, and Willson and to Professor R. W. Carlson for their valuable counsel and services in connection with the investigation. Credit is also due to the following members of the staff of the Engineering Materials Laboratory: S. B. Biddle, Jr., E. H. Brown, Alexander Klein, and C. M. Price.

The United States Bureau of Reclamation and other organizations furnished materials for test.

MATERIALS, APPARATUS, AND METHODS OF TESTING

Materials

The puzzolans and related materials included the following:

Diatomaceous Silicas

Diatomaceous earth (raw, calcined)
Oil-impregnated diatomaceous earth (calcined)
Diatomaceous shale (raw, calcined)

Volcanic Silicas

Fresno pumicite (raw, calcined)
Tuff (raw, calcined)
Italian pozzuolana of Bacoli (raw)
Basaltic tuff (raw, calcined)

Siliceous Rocks

Granite from Arrowrock Dam site (raw, calcined)
Granite C (raw)
Ottawa sand (raw)
Waste asbestos rock (raw)
Quartzite (raw, calcined)

Clays

Clay A (raw, calcined)
Clays B, C, and D (calcined)

Limestones

Limestone dusts A and B (raw)

The portland-cement clinkers were as follows:

Type of Clinker	Compound Composition, Per Cent			
	Tricalcium Silicate	Dicalcium Silicate	Tricalcium Aluminate	Tetracalcium Alumino-ferrite
High-lime	59	22	11	6
Medium-lime	49	32	11	6
Low-lime	37	41	11	9
Low-lime, low-alumina	41	34	8	14

Blending and Grinding

In the portland-puzzolan cements the percentage of cement replacement (by weight) ranged from 10 to 45. The clinkers and puzzolans were interground with an amount of calcined gypsum computed from the equation:

$$\% \; SO_3 = 0.0075 \times (100 + \% \text{ portland-cement clinker in blend})$$

The SO_3 content computed on this basis was 1.1 to 1.5 per cent of the total cement. All cements were ground for 63 min. in a batch ball mill of inside diameter 30 in. and inside length 36 in., operated at 32 r.p.m. The ball charge was 750 lb. of 1½-in. forged steel balls, and the usual charge of material was approximately 80 lb. Prior to grinding, all materials not already passing the No. 8 sieve were crushed to that fineness.

For the portland cements, under the given conditions of grinding the percentage passing the No. 200 sieve ranged from 88 per cent for the low-lime cement to 97 per cent for the medium-lime cement.

Tesis on Cements, Mortars, and Concretes

Following are brief descriptions of the tests for which results are reported herein.

Fineness. The fineness of each cement was determined in terms of the percentage passing the No. 200 sieve (both by dry and by wet sieving), percentage passing the No. 325 sieve (by wet sieving), percentage finer than 10 and 40 microns (by air separation), and the specific surface (surface area in square centimeters per gram of cement). The specific surface of the puzzolans and of the portland-puzzolan cements was determined by a sedimentation method employing a hydrometer. A measure of the specific surface of portland-puzzolan cements was also obtained by means of the Klein turbidimeter[1] and the hydrometer, employing a method made necessary by the fact that the portland-cement and the puzzolanic components of the portland-puzzolan cement differ in opacity, density, etc. The assumption was made that the puzzolanic component of the portland-puzzolan cement is ground to the same fineness as if puzzolan were ground separately for the same length of time.

The values of specific surface reported herein are those determined by hydrometer, computed under the assumptions usually associated with the Riverside microneter and the Klein turbidimeter.

As in this investigation all cements were ground for the same length of time, the more grindable the cement the greater the fineness; this effect is reflected in the tests for strength, volume changes, sulfate resistance, etc.

Chemical and Physical Properties. Complete chemical analysis was made of all puzzolans and portland cement clinkers; and for the portland-puzzolan cements there was determined the SO_3 content, loss on ignition, and insoluble residue. The normal consistency, time of setting, soundness, and specific gravity were determined. All of the cements are sound and are within the standard permissible limits for time of setting.

Activity. The activity of the various puzzolans, or their ability to combine with lime, was determined by compression tests at 7 and 28 days on puzzolan-lime-sand mortar containing 2 parts by weight of puzzolan, 1 part of hydrated lime, and 9 parts of 0 to No. 4 Boulder Dam sand; mortar flow 150 per cent; curing in sealed containers at 70° F. for 12 hr., then at 100° F. for 12 hr., then at 130° F. until 12 hr. before testing, and then at 70° F.

Water Requirement for Fixed Consistency. The relative water requirement of the various cements in neat pastes and concretes was determined by means of trial

[1]"A Suspension Turbidimeter for Determination of Specific Surface of Granular Materials," by Alexander Klein, Proc., A. S. T. M. 1934, Vol. 34, II, pp. 303-321.

batches in which a fixed consistency was obtained, as follows:

1. Normal consistency of neat-cement paste (A. S. T. M. standard method).
2. 140-per cent flow of 1:5.6 concrete containing 0 to ¾-in. Niles gravel aggregate, using fifteen ½-in. drops of a 24-in. flow table.
3. 2½-in. flow of 1:5.65 concrete containing 0 to ¾-in. San Gabriel gravel aggregate, using a dry 6-in. Burmister flow trough.[1]

Heat of Hydration. For selected cements, the rate and total amount of heat of hydration up to the age of 28 days was determined by observations of temperature difference, employing neat-cement specimens in a Carlson vane calorimeter.[2] The vane calorimeter is a calibrated device by means of which the rate of heat generation of the hydrating cement is determined as a function of the temperature differential between the specimen and a surrounding surface which is maintained at constant temperature. The specimen and surface are separated by radial metal vanes. The temperature difference is observed by means of electrical resistance thermometers and a ratio bridge. The heat of hydration is expressed in calories per gram of cement.

For all cements, determinations were made of the heat of immediate hydration, or the heat generated during the first 3 minutes after the cement is wetted by an equal weight of water at 70° F.

Strength. For the various cements, the strength of mortars and concretes was determined by means of tests at various ages up to 1 year, as follows:

1. Tension tests of standard briquets containing standard mortar; standard curing and (in some cases) standard curing for 28 days, followed by storage in air of laboratory.

2. Compression tests of 2 by 4-in. mortar cylinders containing (a) standard mortar, (b) 1:3 mortar containing San Gabriel sand and having a fixed consistency; standard curing and (in some cases) standard curing for 28 days, followed by storage in air of laboratory.

3. Compression tests of 3 by 6-in. concrete cylinders containing 0 to ¾-in. Niles gravel aggregate; cement-aggregate ratio 1:5.6 by weight (1.5 bbl. of cement per cu. yd. of concrete); 140- per cent flow; standard curing.

In connection with the strength tests described in (3), at the age of 1 year the modulus of elasticity, unit weight, and absorption of concrete were determined for all cements.

Volume Changes. For all cements, the expansion of mortar under continued moist storage for 28 days at 70° F. was determined by strain-gage measurements on 1½ by 1½ by 12-in. mortar bars containing Niles sand; cement-aggregate ratio 1:3.25 by weight; water-cement ratio same as for concrete in (3) under "Strength." For selected cements, the expansion under continued moist storage was determined at ages up to 1 year.

For all cements, the specimens, which were standard-cured for 28 days, were employed to determine the contraction of mortar in air of relative humidity 50 per cent at 70° F., beginning at the age of 28 days.

For selected cements, the coefficient of thermal expansion of mortar was determined at ages of 28 days or greater, by observing the length of 1½ by 1½ by 12-in. bars at 130° F. and then reducing the temperature of the bars to 70° F. within 8 hr. and again observing the length.

For selected cements, the expansion of mortar under continued moist storage was determined by strain-gage measurements on 2 by 2 by 18-in. bars of 1:4 mortar con-

[1] "The Concrete Flow Trough," by Donald M. Burmister, Proc., A. S. T. M. 1931, Vol. 31, II, pp. 554-575.

[2] "The Vane Calorimeter," by R. W. Carlson, Proc., A. S. T. M. 1934, Vol. 34, II, pp. 322-328.

taining standard Ottawa sand; water-cement ratio 0.73 by weight.

Resistance to Sulfate Waters. The relative resistance of the various cements to the action of sulfate waters was determined by means of the following tests:

1. Immersion of $\frac{3}{16}$ by 2 by 4-in. neat-cement slabs of water-cement ratio 0.43 by weight in a 10-per cent solution of sodium sulfate, after a 3-day preliminary moist-curing period at 70° F.; specimens inspected daily.

2. Immersion of standard-mortar briquets, after breaking at the age of 28 days in the tension test, in 2 and 10-per cent solutions of sodium sulfate; specimens inspected and weighed at intervals.

3. Immersion of 2 by 2 by 18-in. bars of 1:4 mortar containing standard Ottawa sand (W/C = 0.73 by weight) in a 10-per cent solution of sodium sulfate, after 3 days of preliminary moist curing at 70° F.; length observed at intervals.

4. Immersion of 3 by 6-in. concrete cylinders containing 0 to ¾-in. San Gabriel gravel (cement-aggregate ratio 1:5.65 by weight, 2½-in. flow in Burmister trough) in 1 and 10-per cent solutions of sodium sulfate at 70° F.; also partial immersion of identical specimens outdoors in 1-per cent solution of sodium sulfate; compression test at the age of 7 months; corresponding specimens stored in water under the same conditions of temperature and exposure and tested in compression at the age of 7 months.

Resistance to Freezing and Thawing. The resistance of mortars containing selected cements to the disintegrating action of freezing and thawing was determined by tests on 2 by 4-in. cylinders of 1:3 mortar containing San Gabriel sand and having a water-cement ratio as required to produce a fixed consistency. The specimens were standard-cured for 28 days and then (while continuously saturated) were subjected to 100 alternations of freezing and of thawing at 160° F. The number of cycles (100 or less) was observed at which the specimens attained respectively 2, 25, 50 and 100 per cent of disintegration; and twice the total number of these cycles at all stages was taken as the index of resistance, higher values indicating higher resistance.* The percentage of disintegration was determined as the reduction in length of the cylinder.

TEST RESULTS

The complete test results are too voluminous to be reported herein; hence in general values are given for representative portland-puzzolan cements, for selected tests to determine the various properties, and for selected ages of testing. The discussion applies only under the particular conditions of this investigation with regard to fineness of cement, richness of mix, water-cement ratio, curing temperature, and age at test. In particular, it should be kept in mind (1) that all proportions of materials (including water) are by *weight*, (2) that all cements are ground for an equal period of time and are therefore not of equal fineness, and (3) that in general the water-cement ratios are those which result in fixed consistency of paste, mortar, or concrete and are therefore not the same for the various cements.

In addition to the comparisons of portland-puzzolan cements with one another, generally comparison is made with (1) the portland cement used in the blend, and (2) a portland cement of a type comparable to that of the portland-puzzolan cements. For example, for mass concrete construction either a portland-puzzolan cement (high-lime clinker) or a low-lime portland cement might be used; and these two cements are in this case more nearly comparable than are a portland-puzzolan cement (high-lime clinker) and a high-lime portland cement.

*If any of the four degrees of disintegration was not attained within the 100 cycles, a value of 100 was assigned; thus the maximum possible index was 800.

TABLE 1—CHEMICAL AND PHYSICAL PROPERTIES OF PUZZOLANS

Group	Puzzolan	Raw Or Calcined (1450°F)	Chemical Analysis, Per Cent[a]										Sp. Gr.	Fineness[c]			Activity (Puzzolan-lime-sand Mortar)[d]		
														Per Cent Passing Sieve (Wet)		Spec. Surf., sq. cm per g	W/C, By Wt.	Compressive Strength, p. s. i.	
			SiO_2	Fe_2O_3	Al_2O_3[b]	MnO_2	CaO	MgO	Na_2O	K_2O	SO_3	Ign. Loss		No. 200	No. 325			7 da.	28 da.
Diatomaceous Silicas	Diatomaceous earth	Calc.	72.2	4.1	14.6	0.1	1.3	1.8	2.2	1.6	0.1	2.0	2.24	88.0	77.5	3040	1.10	1340	1420
	Diatomaceous earth	Raw	64.4	3.6	13.0	0.1	1.1	1.6	1.9	1.4	0.1	12.7	2.24	100.0	99.7	2470	1.38	920	1010
	Oil-impregnated diat. earth	Calc.	70.2	4.8	13.8	0.1	3.1	2.4	1.9	1.3	0.5	2.2	2.36	86.0	77.1	3020	0.90	1640	1530
	Diatomaceous shale	Calc.	81.6	3.0	9.5	0.1	0.2	0.8	1.1	1.7	0.3	1.7	2.32	85.5	73.4	2270	0.95	1400	1520
	Diatomaceous shale	Raw	77.2	2.8	9.0	0.1	0.2	0.8	1.0	1.6	0.3	7.0	2.32	88.5	78.0	2010	1.00	1540	1700
Volcanic Silicas	Pumicite	Calc.	74.7	1.4	13.7	0.1	0.7	0.4	1.7	5.6	Tr.	1.2	2.37	98.5	96.0	2860	0.69	490	600
	Pumicite	Raw	72.3	1.4	13.3	0.1	0.7	0.4	1.6	5.4	Tr.	4.2	2.37	99.0	98.3	2950	0.66	640	780
	Tuff	Calc.	72.1	1.4	13.8	0.1	1.6	0.1	3.6	3.5	Tr.	3.7	2.26	79.5	63.0	2060	0.80	550	680
	Tuff	Raw	65.5	1.3	12.6	0.1	1.5	0.1	3.2	3.1	Tr.	12.6	2.26	95.2	86.3	2320	0.88	390	420
	Italian pozzuolana	Raw	58.1	4.4	18.0	0.5	2.4	0.6	4.6	7.0	Tr.	3.9	2.46	97.9	91.1	2520	0.67	740	830
	Basaltic tuff	Calc.	50.6	17.2	16.7	0.7	8.1	5.4	0.4	0.4	0.1	0.4	2.67	92.1	74.5	1610	0.64	—	1870
	Basaltic tuff	Raw	43.3	14.7	14.2	0.6	6.9	4.6	0.3	0.3	0.1	14.8	2.32	82.6	66.9	—	—	—	—
Siliceous Rocks	Granite (Arrowrock Dam)	Calc.	75.0	2.0	13.3	0.1	0.7	0.1	3.8	4.3	Tr.	0.2	2.64	94.0	80.3	—	0.58	50	240
	Granite (Arrowrock Dam)	Raw	75.0	2.0	13.3	0.1	0.7	0.1	3.8	4.3	Tr.	0.2	2.64	95.5	82.4	2050	0.58	50	200
	Granite C	Raw	61.9	5.5	16.8	N.D.	6.2	2.7	N.D.	N.D.	Tr.	2.1	2.75	92.1	78.6	1880	0.63	0	80
	Ottawa sand	Raw	99.9	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.	N.D.	0.1	2.64	89.2	68.0	1540	0.59	0	120
	Waste asbestos rock	Raw	65.5	4.1	2.3	0.1	0.1	15.2	0.5	0.4	0.0	12.2	2.24	88.1	74.1	1930	1.05	240	440
Clays	Clay A	Calc.	60.5	6.9	21.5	0.3	0.9	2.0	2.1	3.0	0.9	2.0	2.64	81.8	72.0	2200	0.65	890	1120
	Clay A	Raw	55.8	6.4	19.8	0.2	0.9	1.8	1.9	2.8	0.9	9.6	2.57	98.4	94.1	1970	0.90	480	740
	Clay B	Calc.	51.5	5.4	14.9	N.D.	15.9	3.0	N.D.	N.D.	0.1	4.1	2.76	94.7	83.4	2360	0.66	310	470
	Clay C	Calc.	63.1	5.0	19.7	N.D.	4.6	2.4	N.D.	N.D.	2.5	0.4	2.54	95.8	85.3	2500	0.66	1200	1560
	Clay D	Calc.	66.5	5.5	17.0	N.D.	3.7	1.7	N.D.	N.D.	0.0	0.4	2.68	92.6	76.0	1930	0.59	310	580
Limestones	Limestone A	Raw	6.9	1.1	2.2	0.1	48.5	2.3	0.4	0.5	Tr.	38.1	2.73	90.9	87.0	2600	0.61	0	0
	Limestone B	Raw	12.3	1.3	4.0	0.1	44.5	1.9	0.9	0.3	0.2	34.5	2.72	92.3	88.3	2590	0.62	0	50

[a]Analysis of calcined puzzolans computed from analysis of raw samples.

[b]Includes small percentages of TiO_2 and P_2O_5.

[c]Ground in batch ball mill for 63 minutes.

[d]2 parts by weight of puzzolan, 1 part of hydrated lime, 9 parts of 0 to No. 4 Boulder Dam sand; puzzolan and lime not interground; 70-100-130°F. curing.

N.D.—Not determined.

Chemical and Physical Properties of Puzzolans

In Table 1 is shown the chemical analysis of the puzzolans. The percentage of silica ranges from 7 to 100; excluding two extremely low values (for the limestones) and one extremely high value (for Ottawa sand) the percentage of silica ranges from 43 to 82. Except for basaltic tuff, which when calcined has an iron-oxide content of 17 per cent, the percentage of iron oxide ranges from 0 to 7. The percentage of alumina ranges from 0 to 22. Excluding the limestones, the percentage of calcium oxide ranges from 0 to 16. Except for waste asbestos rock, which has a magnesia content of 15 per cent, the percentage of magnesia ranges from 0 to 5. In Table 2 are shown the average percentages of the principal oxides in 17 of the puzzolans, by groups.

TABLE 2—PRINCIPAL OXIDES IN PUZZOLANS, BY GROUPS

Puzzolan	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO
Diatomaceous silicas	74.7	4.0	12.6	1.5	1.7
Volcanic silicas	63.9	6.1	15.5	3.2	1.6
Siliceous rocks	75.6	3.4	8.1	1.8	4.5
Clays	60.4	5.7	18.3	6.3	2.3
Limestones	9.6	1.2	3.1	46.5	2.1

The specific gravity of the various puzzolans is shown in Table 1. The specific gravity ranges from 2.24 to 2.76, as compared with 3.16 for a normal portland cement. By groups, the specific gravity is highest for limestones, followed in order by clays, siliceous rocks, volcanic silicas, and diatomaceous silicas.

Grindability of Puzzolans

Although puzzolans are normally interground with portland-cement clinker to form portland-puzzolan cements, the grindability of the separate puzzolans is of interest. The puzzolans employed in the activity tests reported herein were ground for equal periods of time and were of the fineness indicated in Table 1. By groups, the highest specific surface is that for diatomaceous silicas, followed in order by limestones, clays, volcanic silicas, and siliceous rocks.

Activity of Puzzolans

As an indication of puzzolanic activity, or the ability of the various puzzolans to combine with lime, compression tests were made at ages of 7 and 28 days on 2 by 4-in. cylinders of 2:1:9 puzzolan-lime-sand mortar cured in sealed containers at 70° F. for 12 hr., then at 100° F. for 12 hr., and then at 130° F. until 12 hr. prior to the time of test. The results of the tests are given in Table 1. The diatomaceous silicas are most active; the volcanic silicas and clays also show marked activity. The siliceous rocks are relatively inactive. As might be expected, the limestone dusts show no activity in this test.

It has been observed that there is a fair degree of correlation between the results of the activity test on puzzolans and the results of compression tests on standard mortars containing the corresponding portland-puzzolan cements (20 per cent puzzolan, 80 per cent portland cement). The values in Table 3 afford a comparison between the observed compressive strengths of standard mortars at the age of 3 months and compressive strengths computed as follows: To 80 per cent of the 3-month compressive strength of standard mortar containing the high-lime portland cement is added the 28-day compressive strength of the puzzolan-lime-sand mortar.*

With two or three exceptions, in Table 3 the differences between the observed and computed strengths are not large, indicating that in general the activity of a

*Roughly, the 28 days of curing at 70-100-130°F. employed in the activity tests is estimated to be the equivalent of 3 months or more of standard curing.

puzzolan (as measured by the compression test on puzzolan-lime-sand mortar) is an index to the mortar strength exhibited by the corresponding portland-puzzolan cement.

TABLE 3—COMPARISON OF COMPRESSIVE STRENGTHS OF STANDARD MORTAR CONTAINING PORTLAND-PUZZOLAN CEMENTS WITH STRENGTHS COMPUTED FROM RESULTS OF ACTIVITY TEST

Group	Puzzolan	Compressive Strength of Puzzolan-lime-sand Mortar at 28 Days, p. s. i.	Compressive Strength of Standard Mortar Containing Portland-puzzolan Cement (At Age of 3 Months) p. s. i.[a]		
			Computed[b]	Observed	Difference
Diatomaceous Silicas	Diatomaceous earth, calcined	1420	5400	5840	− 440
	Oil-impregnated diatomaceous earth, calcined	1530	5510	6000	− 490
	Diatomaceous shale, calcined	1520	5500	5780	− 280
Volcanic Silicas	Pumicite, calcined	600	4580	4820	− 240
	Pumicite, raw	780	4760	4800	− 40
	Tuff, calcined	680	4660	4970	− 310
	Italian pozzuolana, raw	830	4810	4760	+ 50
	Basaltic tuff, calcined	1870	5850	4500	+1350
Siliceous Rocks	Granite (Arrowrock Dam), raw	200	4180	3900	+ 280
	Granite C, raw	80	4060	3820	+ 240
	Ottawa sand, raw	120	4100	3770	+ 330
	Waste asbestos rock, raw	440	4420	5850	−1430
Clays	Clay A, calcined	1120	5100	4810	+ 290
	Clay A, raw	740	4720	4550	+ 170
	Clay B, calcined	470	4450	4400	+ 50
	Clay C, calcined	1560	5540	4810	+ 730
	Clay D, calcined	580	4560	4260	+ 300
Limestones	Limestone A, raw	0	3980	4360	− 380
	Limestone B, raw	50	4030	4290	− 260

[a]Portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with 80 per cent of high-lime clinker.
[b]28-day compressive strength in activity test plus 80 per cent of 3-month compressive strength of standard mortar containing high-lime portland cement.

Effect of Type of Puzzolan

In Table 4 is shown the general effect of type of puzzolan upon grindability, strength, sulfate resistance, and volume change of portland-puzzolan cements. The values are computed from the data of Tables 9, 13, 17, and 18, and are expressed as percentages of the corresponding value for the high-lime portland cement with which the puzzolans are blended. It should be kept in mind that the comparisons apply only for the fixed percentage of puzzolan (20 per cent), which for some of the puzzolans is not the optimum percentage.

In general, the diatomaceous silicas (the most active puzzolans) exhibit the highest degree of grindability, strength, and resistance to sodium sulfate; but this group also exhibits large contraction of mortar upon drying. The volcanic silicas and clays, which are intermediate with respect to activity, are also intermediate with respect to the properties shown in Table 4. The limestones (of zero activity) show excellent volume constancy and fairly high resistance to sodium sulfate, but low grindability and strength. With the exception of waste asbestos rock, the relatively inactive siliceous rocks show low mortar contraction but also low grindability, strength, and resistance to sodium sulfate.

Effect of Calcination of Puzzolan at 1450° F.

In Table 1 are given all available values of compressive strength of puzzolan-lime-sand mortars for both raw and calcined puzzolans. In Table 5 are shown the same data, with the compressive strengths for the calcined puzzolans expressed as

TABLE 4—EFFECT OF TYPE OF PUZZOLAN UPON PROPERTIES OF PORTLAND-PUZZOLAN CEMENTS

Group	Puzzolan[c]	Percentage of Value for High-lime Portland Cement: Specific Surface[a]	Tens. Str. of Std. Mortar at 3 Mo.[a]	Compr. Str. of Std.-cured Concrete at 1 Yr.[a], [d]	Index of Resistance of Neat Slabs in 10% Na_2SO_4[a]	Contraction of Mortar in Air at 1 Yr.[b]
Diatomaceous Silicas	Diat. earth, calcined	110	131	83	610	149
	Oil-impregnated diat. earth, calcined	110	124	97	606	131
	Diat. shale, calcined	113	129	80	606	156
Volcanic Silicas	Pumicite, calcined	117	121	103	198	118
	Tuff, calcined	104	113	89	491	123
	Italian pozzuolana, raw	111	119	86	121	133
	Basaltic tuff, calcined	93	106	81	89	119
Siliceous Rocks	Granite (Arrowrock Dam), raw	104	86	83	92	113
	Granite C, raw	103	109	71	192	123
	Ottawa sand, raw	97	89	81	80	106
	Waste asbestos rock, raw	111	120	71	615	174
Clays	Clay A, calcined	103	101	83	114	129
	Clay B, calcined	110	105	89	83	114
	Clay C, calcined	107	127	95	332	110
	Clay D, calcined	104	104	82	140	116
Limestones	Limestone A, raw	102	94	82	168	109
	Limestone B, raw	98	107	78	195	106
	None (high-lime p. c.)	100	100	100	100	100

[a]Higher values more desirable.
[b]Lower values more desirable.
[c]All portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.
[d]Cement content of concrete 1.5 bbl. per cu. yd.

TABLE 5—EFFECT OF CALCINATION OF PUZZOLAN AT 1450° F. UPON PROPERTIES OF PUZZOLANS AND PORTLAND-PUZZOLAN CEMENTS

Calcined Puzzolan	Puzzolan: Percentage of Value for Raw Puzzolan: Compr. Str. of Puzzolan-lime-sand Mortar[a] 7 da.	28 da.	Portland-puzzolan Cement[c]: Per Cent of Puzzolan	Percentage of Value for Raw Puzzolan: Specific Surface[a]	Tens. Str. of Std. Mortar at 3 Mo.[a]	Compr. Str. of Std.-Cured Concrete at 1 Yr.[a], [d]	Index of Resistance of Neat Slabs in 10% Na_2SO_4[a]	Contraction of Mortar in Air 1 Yr.[b]
Diatomaceous Silicas								
Diatomaceous earth	146	141	10	103	102	103	147	84
Diatomaceous shale	91	89	30	105	103	95	96	90
Volcanic Silicas								
Pumicite	77	77	20	100	100	99	65	93
Tuff	141	162	30	97	96	120	62	87
Siliceous Rocks								
Granite (Arrowrock Dam)	100	120	20	100	—	—	—	—
Clays								
Clay A	185	151	20	97	101	122	171	80

[a]Higher values more desirable.
[b]Lower values more desirable.
[c]All portland-puzzolan cements contain puzzolan blended with high-lime clinker.
[d]Cement content of concrete 1.5 bbl. per cu. yd.

TABLE 6—CHEMICAL ANALYSIS AND COMPOUND COMPOSITION OF PORTLAND-CEMENT CLINKERS

Clinker	Chemical Analysis, Per Cent									Potential Compound Composition, Per Cent			
	SiO_2	Fe_2O_3	Al_2O_3[a]	CaO	MgO	SO_3	Ign. Loss	Insol. Res.	Free CaO	Tri-Calcium Silicate	Di-Calcium Silicate	Tri-Calcium Aluminate	Tetra-Calcium Alumino-ferrite
High-lime	23.3	1.8	5.2	67.7	1.4	0.2	0.2	0.1	0.7	59	22	11	6
Medium-lime	24.3	1.9	5.2	66.8	1.4	0.1	0.1	0.0	0.2	49	32	11	6
Low-lime	24.0	2.8	5.9	64.5	2.6	1.5[b]	0.2	0.1[b]	0.0	37	41	11	9
Low-lime, low-alumina	22.5	4.5	5.8	63.4	3.1	0.1	0.2	0.0	0.3	41	34	8	14

[a]Includes small percentages of TiO_2 and P_2O_5.
[b]Analysis of cement.

percentages of the strength for raw puzzolan. It is seen that calcination at 1450° F. decreases the activity of diatomaceous shale and pumicite; increases the activity of diatomaceous earth, tuff, and clay A; and affects but little the activity of the granite.

In Table 5 also is shown the effect of calcination of puzzolan upon grindability, strength, sulfate resistance, and volume change of portland-puzzolan cements. For all of the puzzolans, calcination reduces the contraction in air of mortar containing the portland-puzzolan cement. Further tests indicate that the higher the temperature of calcination the less the mortar contraction.

Effect of Type of Portland-Cement Clinker

The chemical analysis and compound composition of the four clinkers are given in Table 6. The effect of type of clinker upon selected properties of portland-puzzolan cements is shown in Table 7; the test results for the portland-puzzolan cements are expressed (in parenthesis) as percentages of the value for the corresponding portland cement. Comparing the portland-puzzolan cements with the corresponding portland cements used in the blend, the relative benefits obtained by blending differ as between the desired properties of grindability, strength, sulfate resistance, and volume constancy upon drying. However, except as regards the contraction of mortar upon drying, it appears that the most favorable relative results are obtained by blending puzzolans with the high-lime clinker.

In the discussions to follow, the conclusions are drawn chiefly with respect to portland-puzzolan cements containing the high-lime portland cement.

Effect of Amount of Puzzolan

The effect of the amount of puzzolan upon selected properties of portland-puzzolan cements containing the high-lime portland cement is shown in Table 8. The test results for portland-puzzolan cements are expressed (in parenthesis) as percentages of the value for high-lime portland cement.

Referring to the values of specific surface in Table 8, it is seen that in general the grindability increases with the amount of puzzolan, although not in direct proportion. Referring to the values of tensile and compressive strength, it is seen that in general the strength is less for greater amounts of puzzolan. For most portland-puzzolan cements, the tensile strength of standard mortar is greater, and the compressive strength of the concrete is less, than that for the high-lime portland cement.* As compared with the low-lime portland cement, on the average the portland-puzzolan cements containing 10 per cent of puzzolan exhibit higher strengths of mortar and concrete, those containing 20 per cent of puzzolan approximately equal strengths, and those containing 30 per cent of puzzolan lower strengths than the corresponding strengths for the low-lime portland cement.

Considering the resistance of neat slabs to the action of sodium sulfate (Table 8), for all of the portland-puzzolan cements except three (those containing Ottawa sand, basaltic tuff, and granite) the resistance is greater than that for the high-lime portland cement; and for five of the active puzzolans the resistance is greater than that for the low-lime portland cement. With few exceptions, the resistance is greater for higher percentages of puzzolan. The corresponding tests on other types of specimen and other concentrations of the sodium-sulfate solution indicate that 10 per cent of puzzolan in portland-puzzolan cement does not greatly improve the sulfate resistance, but that 20 per cent or more of the active puzzolans of diatomaceous or volcanic origin distinctly improves the resistance over that of the corresponding portland

*It must be kept in mind that a rich concrete (1.5 bbl. of cement per cu. yd.) was employed in these tests; further tests, described in part herein, show that portland-puzzolan cements exhibit relatively higher concrete strengths in leaner mixes.

TABLE 7—EFFECT OF TYPE OF PORTLAND-CEMENT CLINKER UPON PROPERTIES OF PORTLAND-PUZZOLAN CEMENTS

Puzzolan	% of Puzz-olan	Spec. Surf., sq. cm per gram[a]	Tens. Str. of Std. Mortar at 3 Mo., p. s. i.[a]	Compr. Str. of Std.-Cured Concrete at 1 Yr., p. s. i.[a], [c]	Index of Resistance of Neat Slabs in 10% Na_2SO_4[a]	Contraction of Mortar in Air at 1 Yr., Millionths per unit[b]
BLENDS WITH HIGH-LIME CLINKER (59% 3C.S, 11% 3C.A)						
None	0	1680	443	6710	114	1236
Diatomaceous shale, calcined	10	(112)	(128)	(91)	(446)	(121)
Pumicite, raw	20	(117)	(121)	(104)	(305)	(127)
Granite (Arrowrock Dam), raw	10	(102)	(104)	(94)	(90)	(109)
BLENDS WITH MEDIUM-LIME CLINKER (49% 3C.S, 11% 3C.A)						
None	0	1560	472	7270	343	1550
Diatomaceous shale, calcined	10	(108)	(112)	(85)	(161)	(112)
Pumicite, raw	10	(111)	(122)	(95)	(50)	(94)
Granite (Arrowrock Dam), raw	10	(112)	(106)	(80)	(44)	(106)
BLENDS WITH LOW-LIME CLINKER (37% 3C.S, 11% 3C.A)						
None	0	1440	470	6230	377	1756
Diatomaceous shale, calcined	10	(108)	(114)	(98)	(192)	(106)
Granite (Arrowrock Dam), raw	10	(105)	——	(86)	——	(103)
BLENDS WITH LOW-LIME, LOW-ALUMINA CLINKER (41% 3C.S, 8% 3C.A)						
None	0	1480	500[d]	5700	688	1603
Diatomaceous shale, calcined	10	(105)	(112)	(102)	(106)	(117)
Pumicite, raw	10	(109)	(97)	(101)	(105)	(116)
Granite (Arrowrock Dam), raw	10	(103)	(87)	(88)	(126)	(104)

[a]Higher values more desirable.
[b]Lower values more desirable.
[c]Cement content of concrete 1.5 bbl. per cu. yd.
[d]Interpolated value.
NOTE—Values in parentheses are percentages of value for corresponding portland cement.

cements. For the percentages shown, the portland-puzzolan cements containing limestone are more resistant than the high-lime portland cement and are more resistant than the portland-puzzolan cements containing Ottawa sand, granite, basaltic tuff, and clay A.

Considering the values of contraction of mortar in air (Table 8), it is seen that in all cases (1) the contraction is greater than that for the high-lime portland cement, and (2) the higher the percentage of puzzolan, the greater the contraction. Comparing the portland-puzzolan cements with the low-lime portland cement, it is seen that with two exceptions (diatomaceous earth and shale, 20 per cent) the contraction is less for portland-puzzolan cements containing 10 and 20 per cent of puzzolan than for the low-lime cement. For six of the portland-puzzolan cements containing 30 per cent of puzzolan, the contraction is equal to or less than that for the low-lime cement.

The optimum percentage of a particular puzzolan necessarily depends upon the property of mortar or concrete under consideration, as well as upon the type of clinker. On the whole, consideration of the foregoing properties indicates that the optimum percentage is likely to lie within the limits of 10 to 30 per cent. In the discussions to follow, except as noted the conclusions are drawn with respect to portland-puzzolan cements containing 20 per cent of puzzolan.

TABLE 8—EFFECT OF AMOUNT OF PUZZOLAN UPON PROPERTIES OF PORTLAND-PUZZOLAN CEMENTS

Group	Puzzolan[c]	% of Puzz-olan	Spec. Surf. sq. cm. per gram[a]	Tens. Str. of Std. Mortar at 3 Mo., p. s. i.[a]	Compr. Str. of Std.-Cured Concrete at 1 Yr., p. s. i.[a, d]	Index of Resistance of Neat Slabs in 10% Na_2SO_4[a]	Contraction of Mortar in Air at 1 Yr., Millionths per unit[b]
Portland Cements	None (high-lime portland cement)	0	1680	443	6710	114	1236
Diatomaceous Silicas	Diatomaceous earth, calcined	10 20	(106) (110)	(117) (131)	(93) (83)	(263) (610)	(116) (149)
	Oil-impregnated diatomaceous earth, calcined	10 20 30	(105) (110) (118)	(120) (124) (128)	(100) (97) (85)	(274) (606) (622)	(116) (131) (160)
	Diatomaceous shale, calcined	10 20 30	(112) (113) (109)	(128) (129) (123)	(91) (80) (81)	(446) (606) (607)	(121) (156) (185)
Volcanic Silicas	Pumicite, calcined	20 30	(117) (124)	(121) (128)	(103) (101)	(198) (611)	(118) (142)
	Tuff, calcined	20 30	(104) (110)	(113) (111)	(89) (83)	(491) (398)	(123) (161)
	Basaltic tuff, calcined	20 30	(93) (93)	(106) (105)	(81) (69)	(89) (99)	(119) (122)
Siliceous Rocks	Granite (Arrowrock Dam), raw	10 20 30	(102) (104) (105)	(104) (86) (81)	(94) (83) (71)	(90) (92) (98)	(109) (113) (116)
	Ottawa sand, raw	20 30	(97) (101)	(89) (83)	(81) (73)	(80) (75)	(106) (110)
Clays	Clay A, calcined	20 30	(103) (107)	(101) (110)	(83) (73)	(114) (118)	(129) (140)
Limestones	Limestone A, raw	20 30	(102) (103)	(94) (89)	(82) (70)	(168) (241)	(109) (114)
Portland Cements	None (low-lime portland cement)	0	(86)	(106)	(93)	(331)	(142)

[a]Higher values more desirable.
[b]Lower values more desirable.
[c]All portland-puzzolan cements contain puzzolan blended with high-lime clinker.
[d]Cement content of concrete 1.5 bbl. per cu. yd.
NOTE—Values in parentheses are percentages of value for high-lime portland cement.

Chemical and Physical Properties of Portland-Puzzolan Cements

In Table 9 are shown the chemical and physical properties of seventeen portland-puzzolan cements containing 20 per cent of puzzolan blended with high-lime clinker. Corresponding values are shown for the four portland cements included in the investigation.

The specific gravity of the portland-puzzolan cements in Table 9 ranges from 2.91 to 3.07, as compared with 3.16 for the high-lime portland cement with which the puzzolans are blended. All of the portland-puzzolan cements tested in this investigation are sound and exhibit normal times of setting.

The specific gravity of the diatomaceous silicas and of certain volcanic silicas, which in portland-puzzolan cements exhibit the best strength and sulfate resistance, is considerably lower than that of the remaining puzzolans. Further, as between the various clays or as between the various siliceous rocks, the lower the specific gravity the better the strength and sulfate resistance. However, the relation between specific gravity and performance is not entirely consistent throughout the range of specific gravity for all puzzolans.

TABLE 9—CHEMICAL AND PHYSICAL PROPERTIES OF PORTLAND AND PORTLAND-PUZZOLAN CEMENTS

Group	Puzzolan[f]	Chemical Analysis of Cement, Per Cent			Sp. Gr.	Fineness[b]: Per Cent Passing Sieve (Wet)		Fineness[b]: Per Cent Finer Than Diam.		Fineness[b]: Spec Surf., sq. cm per g			Time of Setting (Gillmore) hr.: min.		Water Requirement		
		SO_3	Ign. Loss	Insol. Res.		No. 200	No. 325	40 Microns	10 Microns	Puzzolan Component[g]	Portland-Cement Component[h]	Portland-Puzzolan Cement[g]	Init.	Final	Norm. Cons., Per Cent Water[c]	W/C of Concrete, by Wt.: 140% Flow[d]	W/C of Concrete, by Wt.: 2½-in. Flow[e]
Diatomaceous Silicas	Diatomaceous earth, calcined	1.5	0.8	13.8	2.97	94.0	83.5	78.7	35.3	3040	1540	1840	2:48	6:17	31.5	0.59	—
	Oil-impregnated diat. earth, calc.	1.7	0.6	13.0	3.00	96.3	85.0	80.0	41.3	3020	1550	1840	2:45	4:55	30.5	0.58	0.60
	Diatomaceous shale, calcined	1.4	0.9	14.2	2.99	96.3	86.6	78.3	38.5	2270	1800	1890	3:05	5:32	32.3	0.61	0.67
Volcanic Silicas	Pumicite, calcined	1.4	0.5	17.6	2.96	98.5	89.8	81.1	38.1	2860	1740	1960	2:44	6:04	24.6	0.52	0.58
	Tuff, calcined	1.4	1.2	15.6	2.94	95.3	84.0	75.7	34.3	2060	1660	1740	3:00	5-48	26.3	0.53	0.58
	Italian pozzuolana, raw	1.3	1.7	10.9	2.98	98.3	91.0	83.3	37.6	2520	1690	1860	3:17	5:52	26.4	0.56	0.58
	Basaltic tuff, calcined	1.4	0.9	14.1	3.05	95.2	82.9	72.1	29.3	1610	1550	1560	2:23	4:43	24.3	0.55	0.57
Siliceous Rocks	Granite (Arrowrock Dam), raw	1.4	0.5	18.3	3.06	94.5	76.8	75.1	34.1	2050	1680	1750	2:47	5:17	24.0	0.51	0.56
	Granite C, raw	1.4	0.6	15.6	3.06	95.9	83.3	74.7	34.1	1880	1690	1730	2:39	5:14	24.5	0.55	—
	Ottawa sand, raw	1.3	0.4	19.3	3.06	93.5	82.4	72.3	32.5	1540	1650	1630	2:50	5:05	23.0	0.49	0.57
	Waste asbestos rock, raw	1.3	2.5	4.1	2.91	97.2	86.2	80.6	37.7	1930	1860	1870	2:55	5:45	32.5	0.63	0.63
Clays	Clay A, calcined	1.5	0.7	14.0	3.06	94.0	82.8	77.5	38.0	2200	1610	1730	3:02	6:17	26.0	0.55	0.58
	Clay B, calcined	1.4	0.8	13.9	3.07	96.4	87.0	77.2	35.2	2360	1720	1850	2:50	4:59	24.0	0.54	0.56
	Clay C, calcined	1.5	0.6	17.7	3.02	97.1	86.4	79.5	36.4	2500	1630	1800	2:32	4:27	25.0	0.54	0.57
	Clay D, calcined	1.4	0.4	18.1	3.05	96.8	85.9	76.7	33.8	1930	1710	1750	3:20	4:50	23.5	0.53	0.57
Limestones	Limestone A, raw	1.4	7.8	1.9	3.06	91.3	80.6	73.3	35.9	2600	1500	1720	3:03	5:33	22.0	0.53	—
	Limestone B, raw	1.4	7.2	3.5	3.06	91.7	80.0	73.9	36.4	2590	1400	1640	3:16	5:36	23.5	0.54	—
Portland Cements	None (high-lime p.c.)	0.2[a]	0.2[a]	0.1[a]	3.16	95.7	86.6	76.2	32.4	—	1680	1680	2:25	4:20	22.0	0.52	0.57
	None (medium-lime p.c.)	0.1[a]	0.1[a]	0.0[a]	3.18	96.5	87.1	75.5	31.2	—	1560	1560	2:29	5:24	23.0	0.51	0.58
	None (low-lime p.c.)	1.5	0.2[a]	0.1	3.21	89.5	76.1	63.8	28.5	—	1440	1440	3:00	5:20	21.5	0.54	—
	None (low-lime, low-alumina p.c.)	0.1[a]	0.2[a]	0.0[a]	3.22	92.2	79.4	67.3	29.5	—	1480	1480	3:01	5:46	22.0	0.57	0.56

[a]Analysis of clinker.
[b]Ground in batch ball mill for 63 minutes.
[c]A. S. T. M. standard test on neat-cement paste.
[d]Flow-table test on 1:5.6 concrete containing 0 to ¾-in. Niles gravel.
[e]Burmister-flow-trough test on 1:5.65 concrete containing 0 to ¾-in. San Gabriel gravel.
[f]All portland-puzzolan cements contain 20 per cent of puzzolan by weight, blended with high-lime clinker.
[g]By hydrometer.
[h]Computed from specific surfaces of puzzolanic component and portland-puzzolan cement.
NOTE—All cements sound in A. S. T. M. standard steam test on neat-cement pat.

TABLE 10—FINENESS OF CEMENTS GROUND FOR EQUAL PERIODS, BY GROUPS

Puzzolan[a]	Percentage of Value for High-lime Portland Cement							
	Sp. Gr.	Amount Passing Sieve		Amount Finer Than Diam.		Specific Surface		
		No. 200	No. 325	40 Microns	10 Microns	Puzzolan Component	Portland-Cement Component	Portland-Puzzolan Cement
Diatomaceous silicas	95	100	98	104	118	165	97	111
Volcanic silicas	94	101	100	102	107	135	99	106
Siliceous rocks	96	100	95	99	107	110	102	104
Clays	97	100	99	102	111	134	99	106
Limestones	97	96	93	97	112	154	86	100
None (high-lime port. cement)	100	100	100	100	100	—	100	100
None (low-lime port. cement)	101	94	88	84	88	—	86	86

[a]All portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.

Fineness; Grindability of Cements

As all of the cements were ground for equal periods of time, their fineness is an indication of relative grindability. In Table 9 are shown values of fineness expressed as percentages passing the No. 200 and No. 325 sieves, as percentages finer than 40 and 10 microns, and in terms of specific surface. With few exceptions, the percentage passing the No. 325 sieve and the percentages finer than 40 and 10 microns show excellent correlation with the relative values of specific surface.

Referring to the values of specific surface of portland cements in Table 9, it is seen that the high-lime cement is the easiest to grind and that the low-lime cement is the most difficult to grind.

Referring to the values of specific surface of portland-puzzolan cements in Table 9, it is seen that all of these cements are more grindable than the medium-lime and low-lime portland cements, and that all except three (those containing basaltic tuff, Ottawa sand, and limestone B) are more grindable than the high-lime portland cement with which the puzzolans are blended.

In Table 9 is shown the specific surface of the puzzolanic component of the portland-puzzolan cements, based upon the assumption that the grinding characteristics of the puzzolan are the same when interground with clinker as when ground separately. (As the puzzolanic component is only a fraction of the total cement, a fairly large error in the assumption would affect the computed specific surface of the portland-cement component but little.) It is seen that the two portland-puzzolan cements which are most difficult to grind, as indicated by their low specific surfaces for equal periods of grinding, contain the two puzzolans which are most difficult to grind (basaltic tuff and Ottawa sand).

It is possible that the fineness of the portland-cement component of a portland-puzzolan cement has greater significance than the fineness of the total (blended) cement as reported herein. In Table 9, the computed specific surface of the portland-cement component ranges from 1400 to 1860 sq. cm. per gram, averaging 1650 sq. cm. per gram as compared with 1680 sq. cm per gram for the high-lime portland cement ground separately. These differences are not great enough to permit evaluation of the effect of fineness of portland-cement component upon the properties of portland-puzzolan cements.

In Table 10 are shown the fineness data of Table 9, averaged by groups of puzzolans and with the values expressed as percentages of the corresponding value for the high-lime portland cement. With the exception of the group containing the limestones, the portland-puzzolan cements have on the average a specific surface of portland-

cement component approximately equal to that of the high-lime portland cement; therefore their higher fineness (specific surface) appears to be due to the higher fineness of the puzzolanic component. The highest degree of fineness is attained by the group of portland-puzzolan cements containing diatomaceous silicas, and the lowest by the group containing limestones.

As shown in Table 5, calcination of the puzzolan at 1450° F. has but little effect upon the grindability of the six portland-puzzolan cements there listed.

A comparison of the specific surface of portland-puzzolan cements containing different amounts of puzzolan up to 30 per cent (Table 8) indicates that in general the grindability increases with the percentage of puzzolan. An exception is diatomaceous shale, for which the maximum grindability is for portland-puzzolan cement containing 25 per cent of puzzolan.

Water Requirement for Fixed Consistency

Table 11 shows the water requirement of the various groups of puzzolans for fixed consistencies which were employed as the bases for specimen manufacture. The values for 20 per cent of puzzolan are complete averages from Table 9, but the data for other percentages of puzzolan are incomplete, as tests were not made on all pozzolans in all percentages.

In general, the portland-puzzolan cements require more water than does the high-lime portland cement. The greater the percentage of puzzolan, the greater the water requirement. The greatest amounts of water are required by the diatomaceous silicas and waste asbestos rock, and the least by Ottawa sand. In general, the portland-puzzolan cements of higher fineness require more water to produce the given consistency.

As the water-cement ratios determined in these tests were employed in the manufacture of specimens for other tests, the effect of water requirement is reflected in the results of tests to determine strength, volume change, sulfate resistance, etc.

TABLE 11—RELATIVE WATER REQUIREMENT OF PORTLAND-PUZZOLAN CEMENTS FOR FIXED CONSISTENCY, BY GROUPS

Puzzolan Blended With High-lime Clinker in Percentage Indicated	Percentage of Value for High-lime Portland Cement								
	Normal Consistency of Neat-cement Paste			140 Per Cent Flow of Concrete (Flow Table)			2½-in. Flow of Concrete (Burmister Trough)		
	10%	20%	30%	10%	20%	30%	10%	20%	30%
Diatomaceous silicas	121	143	167	106	114	115	105	111	126
Volcanic silicas	—	116	120	—	104	107	—	102	—
Siliceous rocks	—	118	—	—	105	—	—	103	—
Clays	—	112	120	—	104	106	—	100	105
Limestones	—	103	107	—	103	104	—	—	—

Water Gain of Concretes

In a separate series of tests, the water gain up to the time of initial set was determined on 18 by 36-in. cylindrical specimens of concrete containing (a) a medium-lime, low-alumina portland cement, (b) a low-lime, low-alumina portland cement, and (c) a portland-puzzolan cement consisting of the medium-lime, low-alumina portland cement blended with 25 per cent of calcined diatomaceous shale. The test results are shown in Table 12. It is seen that the portland-puzzolan cement exhibits under the conditions of these tests a greater water-retaining capacity than either of the portland cements.

Permeability of Concretes

In the same separate series of tests, the permeability of 30 by 30-in. cylindrical concrete specimens was determined at the age of 28 days, in terms of the total inflow of water under 100 p.s.i. pressure applied through a porous mortar block at the center of the cylinder. The test results are given in Table 12. It is seen that on the average the portland-puzzolan cement exhibits greater resistance to the percolation of water.

Strength of Mortars and Concretes

In Table 13 are given values of strength of mortars and concretes at various ages up to 1 year. In Table 14 are given certain of these values, by groups of puzzolans, expressed as percentages of the corresponding values for high-lime portland cement.

TABLE 12—WATER GAIN AND PERMEABILITY OF CONCRETES

Cement	Avg. Spec. Surf., sq. cm per gram	Average Water Gain on 18 by 36-in. Concrete Cylinder Up to Time of Initial Set, ml		Average Total Inflow of Water Into 30 by 30-in. Concrete Cylinder Over Period of 36 Hours at 100 p. s. i., ml[b]	
		0.8 bbl. per cu. yd.	1.0 bbl. per cu. yd.	0.8 bbl. per cu. yd.	1.0 bbl. per cu. yd.
Medium-lime, low-alumina portland cement	1950	195	76	790	180
Low-lime, low-alumina portland cement	1980	86	40	440	200
Portland-puzzolan cement[a]	2130	31	28	230	120

[a]Medium-lime, low-alumina clinker blended with 25 per cent of calcined diatomaceous shale.
[b]Permeability specimens standard-cured 28 days before test.
NOTE—Concrete contains 0 to 6-in. gravel; slump 1½ in. Values are average for two finesses of cement (specific surface nominally 1800 and 2200 sq. cm per gram).

Standard Mortar. As shown in Table 14, at the age of 3 months the highest tensile strength of standard-cured standard mortar is attained by the group of portland-puzzolan cements containing the diatomaceous silicas, followed in order by the volcanic silicas, clays, limestones, and siliceous rocks. The tensile strength is higher for all groups of portland-puzzolan cements than for the high-lime portland cement; and is higher for the groups containing the active puzzolans than for the low-lime portland cement.

Considering the compressive strength of standard-cured standard mortar at the age of 3 months (Table 14), it is seen that the strength is lower for four groups of portland-puzzolan cements than for the high lime portland cement, but is higher for the group containing diatomaceous silicas than for the high-lime portland cement.

Incomplete results of the 1-year tension and compression tests on standard mortar indicate that the groups of portland-puzzolan cements remain in the same order with regard to strength as at the age of 3 months, but that for the volcanic silicas the strengths approach those for the diatomaceous silicas.

Considering the tensile strength of standard mortar standard-cured for 28 days and then stored in air to the age of 3 months (Table 13), it is seen that under these conditions of air storage the strength is higher for all but three of the portland-puzzolan cements than for the high-lime portland cement; and that the strength is considerably higher for all portland-puzzolan cements than for the low-lime portland cement. Comparing the groups of portland-puzzolan cements (Table 14), the strength after air storage is highest for the diatomaceous silicas and lowest for the siliceous rocks.

TABLE 13—STRENGTH AND ELASTICITY OF STANDARD-CURED MORTARS AND CONCRETES CONTAINING PORTLAND AND PORTLAND-PUZZOLAN CEMENTS

Group	Puzzolan[g]	Spec. Surf., sq. cm per gram	Standard Mortar							Plastic Mortar[b]		Concrete (1.5 bbl. per cu. yd.)[c]						
			Water-Cement Ratio, by wt.	Tensile Strength, p. s. i.					Compr. Str., p. s. i.	W/C, by wt.	Compr. Str., p. s. i.	Water-Cement Ratio, by wt.	Compressive Strength, p. s. i.			Secant Mod. of Elast.[d]	Absorption, Per Cent by wt.	Unit Weight[e] lb./cu. ft.
				Water Storage				Air Storage[a]										
				7 da.	28 da.	3 mo.	1 yr.	3 mo.	3 mo.		3 mo.		28 da.	3 mo.	1 yr.	1 yr.	1 yr.	1 yr.
Diatomaceous Silicas	Diatomaceous earth, calc.	1840	0.47	354	513	580	557	625	5840	—	—	0.59	4350	5250	5580	4.54	7.6	150.0
	Oil-impregnated diat. earth, calc.	1840	0.46	407	533	548	582	556	6000[f]	0.58	5950	0.58	5190	6020	6540	4.44	7.5	152.0
	Diatomaceous shale, calcined	1890	0.47	382	513	571	526	549	5780	0.66	5200	0.61	4650	5160	5400	4.82	8.1	150.0
Volcanic Silicas	Pumicite, calcined	1960	0.42	339	479	536	593	535	4820	0.56	5650	0.52	4630	6020	6940	5.08	7.2	152.0
	Tuff, calcined	1740	0.44	314	491	502	542	482	4970	0.56	5550	0.53	5080	5830	5950	5.12	6.7	152.1
	Italian pozzuolana, raw	1860	0.44	386	480	528	—	530	4760	0.56	5340	0.56	4000	4700	5780	4.38	7.5	150.0
	Basaltic tuff, calcined	1560	0.42	341	423	470	—	561	4500[f]	0.53	5400	0.55	4190	4660	5420	4.61	7.7	148.6
Siliceous Rocks	Granite (Arrowrock Dam), raw	1750	0.42	312	380	380[f]	396[f]	495[f]	3900[f]	0.52	4910	0.51	4620	5110	5580	4.44	5.7	152.6
	Granite C, raw	1730	0.42	335	411	481	—	518	3820	—	—	0.55	3890	4520	4750	4.28	7.3	150.2
	Ottawa sand, raw	1630	0.41	310	413	395	370	493	3770	0.54	4470	0.49	4300	5090	5400	4.74	5.9	153.5
	Waste asbestos rock, raw	1870	0.47	453	566	530	523	545	5850	0.61	4950	0.63	3990	4380	4750	3.90	9.1	148.0
Clays	Clay A, calcined	1730	0.43	367	446	447	460	549	4810	0.56	5360	0.55	4350	5190	5540	5.00	6.8	152.0
	Clay B, calcined	1850	0.42	374	468	464	473	580	4400	0.54	5390	0.54	4350	5010	5940	4.86	6.7	151.5
	Clay C, calcined	1800	0.43	379	468	561	552	521	4810	0.54	6000	0.54	4450	5810	6340	4.60	6.3	151.5
	Clay D, calcined	1750	0.42	361	422	463	465	562	4260	0.54	5220	0.53	4050	5100	5520	4.60	6.6	152.0
Limestones	Limestone A, raw	1720	0.41	364	414	418	406	559	4360	—	—	0.53	4170	4890	5500	5.24	6.6	152.7
	Limestone B, raw	1640	0.42	368	421	476	—	539	4290	—	—	0.54	3730	4580	5250	4.60	7.5	150.5
Portland Cements	None (high-lime p.c.)	1680	0.41	368	447	443	404	509	4970	0.54	5110	0.52	5370	6210	6710	5.30	6.1	154.0
	None (medium-lime p.c.)	1560	0.41	370	448	472	446	503	5180	0.56	6530	0.51	5980	6570	7270	5.94	5.7	154.0
	None (low-lime p.c.)	1440	0.40	331	422	470	—	438	—	—	—	0.54	3700	5480	6230	5.00	6.0	153.0
	None (low-lime, low-alumina p.c.)	1480	0.41	335	447	500[f]	—	—	—	0.54[f]	6180[f]	0.57	4230	5410	5700	4.39	6.6	148.6

[a] In air of laboratory after 28 days of standard curing; tested dry.
[b] Cement-aggregate ratio 1:3 by weight; 0 to No. 4 San Gabriel sand; 2 by 4-in. cylinders.
[c] Cement-aggregate ratio 1:5.6 by weight; 0 to ¾-in. Niles gravel; 3 by 6-in. cylinders.
[d] At 25 per cent of the ultimate strength; millions of pounds per square inch.
[e] Saturated.
[f] Interpolated value.
[g] All portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.

TABLE 14—STRENGTH OF STANDARD-CURED MORTARS AND CONCRETES, BY GROUPS OF PORTLAND-PUZZOLAN CEMENTS

Puzzolan[a]	Percentage of Value for High-lime Portland Cement						
	Standard Mortar				Concrete (1.5 bbl. per cu. yd.)		
	Tensile Strength			Compr. Str.	Compr. Str.		Elastic Modulus
	Water Storage		Air Storage[b]				
	3 mo.	1 yr.	3 mo.	3 mo.	3 mo.	1 yr.	1 yr.
Diatomaceous silicas	128	137	113	118	88	87	87
Volcanic silicas	115	—	103	96	85	90	91
Siliceous rocks	101	—	101	87	77	76	82
Clays	109	121	109	92	85	87	91
Limestones	101	—	108	87	76	80	93
None (high-lime portland cement)	100	100	100	100	100	100	100
Ditto, less 20% cement in mix)	—	—	—	—	69	71	86
None (low-lime portland cement)	106	—	86	—	88	93	94

[a]Portland-puzzolan cements contain 20 per cent of puzzolan, blended with high-lime clinker.
[b]In air of laboratory after 28 days of standard curing.

TABLE 15—EFFECT OF RICHNESS OF MIX UPON RELATIVE COMPRESSIVE STRENGTHS OF STANDARD-CURED CONCRETES CONTAINING PORTLAND AND PORTLAND-PUZZOLAN CEMENTS

Cement	Spec. Surf., sq. cm per gram	Compressive Strength of Concrete at 3 Months, p. s. i.			Percentage of Value for Corresponding Portland Cement		
		0.8 bbl. per cu. yd.[c]	1.0 bbl. per cu. yd.[c]	1.5 bbl. per cu. yd.[d]	0.8 bbl. per cu. yd.[c]	1.0 bbl. per cu. yd.[c]	1.5 bbl. per cu. yd.[d]
Medium-lime portland cement	1560	—	—	6570	—	—	100
Portland-puzzolan cement[a]	1770	—	—	5250	—	—	80
Medium-lime, low-alumina portland cement	1790 2100	1820 2490	3210 3980	— —	100 100	100 100	— —
Portland-puzzolan cement[b]	2000 2250	2760 2740	3630 3950	— —	152 110	113 99	— —

[a]Medium-lime portland cement blended with 20 per cent of calcined diatomaceous shale.
[b]Medium-lime, low-alumina portland cement blended with 25 per cent of calcined diatomaceous shale.
[c]6 by 12-in. cylinders; 0 to 1½-in. gravel; slump ¾ in.
[d]3 by 6-in. cylinders; 0 to ¾-in. gravel; flow 140 per cent.

Plastic Mortar. In general, at the age of 3 months the compressive strengths of the plastic mortars containing San Gabriel sand (Table 13) are about 20 per cent higher than the strengths of corresponding standard mortars. However, for the three puzzolans having the highest water requirement to produce a fixed consistency, the strength of the plastic mortar is less than that of the standard mortar.

Concrete. As shown in Table 13, in general the compressive strengths of concrete containing 1.5 bbl. of cement per cu. yd. are at all ages up to 1 year less for the portland-puzzolan cements than for the high-lime portland cement; and are at the ages of 3 months and 1 year less for the portland-puzzolan cements than for the low-lime portland cement. Comparing the various groups of portland-puzzolan cements (Table 14), the compressive strength of concrete is relatively higher for the groups of concretes containing the active puzzolans (diatomaceous silicas, volcanic silicas, and clays).

It is emphasized that the concrete employed in these tests is relatively rich. Other tests have shown that the relative strengths for portland-puzzolan cements are higher in leaner mixes of concrete. The effect of richness of mix is shown in Table 15, for concrete containing respectively 0.8, 1.0, and 1.5 bbl. of cement per cu. yd.* It is seen that the leaner the mix, the higher the ratio of compressive strength for portland-puzzolan cements to the strength for the portland cements with which the puzzolans are blended.

Referring again to Table 14, it is seen that even the inactive puzzolans (siliceous rocks and limestones) exhibit higher compressive strengths of concrete than does the high-lime portland cement less 20 per cent of cement in the mix. The difference in strength is probably due in large part to the fact that these puzzolans serve as workability agents and void fillers, thus reducing the water requirement.

Elasticity of Concretes

In Table 13 are given values of secant modulus of elasticity of concrete at the age of 1 year; and in Table 14 these values are expressed, by groups, as percentages of the value for high-lime portland cement. In general, the moduli are proportional to the compressive strengths at the same age. For the portland-puzzolan cements, the modulus ranges from 3.90 to 5.24 million lb. per sq. in.; the average value is 4.66 million lb. per sq. in., or 88 per cent of the value for the high-lime portland cement.

Absorption of Concretes

In Table 13 is shown the absorption of concrete at the age of 1 year. For all cements, the absorption ranges from 5.7 to 9.1 per cent by weight, indicating no pronounced difference as between cements in this respect. The greatest absorption is exhibited by the portland-puzzolan cements containing waste asbestos rock and diatomaceous shale.

Unit Weight of Concretes

Values of unit weight of concrete are shown in Table 13. The unit weight of the concretes containing portland-puzzolan cements ranges from 0.5 to 6.0 lb. per cu. ft. less than that for the high-lime portland cement, the average difference being 2.9 lb. per cu. ft. or about 2 per cent.

Heat of Hydration of Cements

Fig. 1 shows the rate of heat generation of several neat cements during the first 24 hours of hydration. For the high-lime portland cement, the maximum rate occurs at the age of approximately 11 hours. The rate for the portland-puzzolan cements containing active puzzolans (diatomaceous shale and pumicite) reaches a maximum at an earlier age than that for the portland cement; and the maximum rate for the inactive material (limestone) reaches a maximum at about the same age. For portland-puzzolan cements containing the greater percentages of diatomaceous shale, the maximum rate of heat generation is smaller in amount and occurs at an earlier age. The area under each curve represents the total heat of hydration up to the age of 1 day.

In the tabulation shown in Fig. 1 are given values of (1) the heat of immediate hydration, and (2) the total heat of hydration up to ages of 1, 3, 7, and 28 days. The total heat of hydration at all ages is less for the portland-puzzolan cements than for the high-lime portland cement, the difference being roughly proportional to the percentage of puzzolan.

The total heat of hydration of the low-lime cement up to the age of 28 days is 80.7 calories per gram, a value approximately equal to the average value for the six portland-puzzolan cements (listed in Fig. 1) which contain 20 per cent of puzzolan.

*The concretes containing 0.8 and 1.0 bbl. of cement per cu. yd. were tested in a separate investigation.

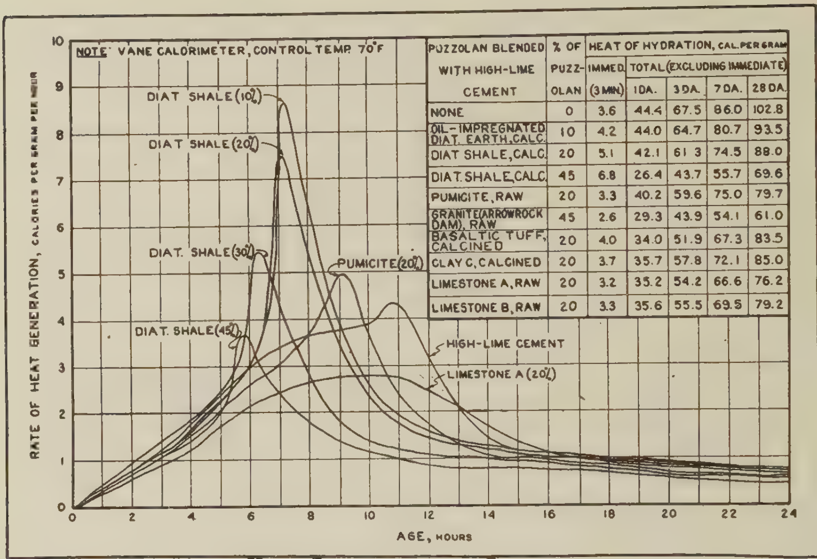

FIG. 1—HEAT GENERATION OF NEAT CEMENTS

In general, it has been observed that portland-puzzolan cements generate a relatively large portion of their total heat of hydration at the very early ages.

TABLE 16—STRENGTH-HEAT RATIO OF PORTLAND AND PORTLAND-PUZZOLAN CEMENTS

Pozzolan[a]	Tensile Strength of Standard Mortar at 28 Days, p. s. i.	Total Heat of Hydration to 28 Days, cal. per gram	Strength-Heat Ratio
Diatomaceous shale, calcined	513	88.0	5.8
Pumicite, raw	430	79.7	5.4
Basaltic tuff, calcined	423	83.5	5.1
Clay C, calcined	468	85.0	5.5
Limestone A, raw	414	76.2	5.4
Limestone B, raw	421	79.2	5.3
None (high-lime portland cement)	447	102.8	4.4
None (low-lime portland cement)	422	80.7	5.2

[a]Portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.

The ratio of strength to heat generation up to a given age is sometimes taken as a measure of the efficiency of cements for mass concrete construction. For the six portland-puzzolan cements containing 20 per cent of puzzolan, listed in Fig. 1, the strength-heat ratios at 28 days based upon the tensile strength of standard mortar are shown in Table 16, together with corresponding values for the high-lime and low-lime portland cements. The strength-heat ratios are considerably higher for all portland-puzzolan cements than for the high-lime portland cement, and with one exception are higher for the portland-puzzolan cements than for the low-lime portland cement.

It has been observed that the heat of immediate hydration is greater for finer portland-puzzolan cements and for those containing the more active puzzolans.

Resistance to Action of Sodium Sulfate

The relative resistance of cements to the action of sodium-sulfate solutions is ndicated by the values for certain of the tests, shown in Table 17.

Neat-Cement Slabs. Considering the values of "index of resistance" for neat-cement slabs in a 10-per cent solution of sodium sulfate (Table 17), it is seen that with three exceptions (Ottawa sand, Clay B, and basaltic tuff) the portland-puzzolan cements are more resistant than the high-lime portland cement with which they are blended. Of the portland-puzzolan cements, the highest resistance is exhibited by those containing the diatomaceous silicas, waste asbestos rock, and tuff. Of the portland cements, the low-lime, low-alumina cement is most resistant and the high-lime cement is least resistant.

Standard Mortar Briquets. As shown in Table 17, a high degree of resistance of standard mortar briquets to the action of 2 and 10-per cent solutions of sodium sulfate is exhibited by the portland-puzzolan cements containing the diatomaceous silicas, pumicite, tuff, the limestones, and waste asbestos rock. The least resistant portland-puzzolan cements are those containing the granites, Ottawa sand, and Clay A. Of the portland cements, the low-lime, low-alumina cement is most resistant and the high-lime cement is least resistant. In general, the portland-puzzolan cements containing the diatomaceous silicas, volcanic silicas, and limestones are more resistant than the high-lime portland cement; and some of the portland-puzzolan cements (blended with high-lime clinker) are more resistant than any of the portland cements.

Expansion of Mortar. In this test, the action of a 10-per cent solution of sodium sulfate on mortar bars is indicated by the difference in expansion between bars immersed in the sulfate solution and corresponding bars immersed in water, greater differences indicating lower resistance (Table 17). Prior to the age of 2 months, very high expansions were attained by the high-lime portland cement and by the portland-puzzolan cements containing Ottawa sand, granite, and clay A; and the tests on these cements were discontinued. Relatively high resistance (as indicated by low values of expansion) is exhibited by the portland-puzzolan cements containing the diatomaceous silicas, waste asbestos rock, tuff, and pumicite.

Compressive Strength of Concrete. In this test, 3 by 6-in. cylinders of concrete containing 1.5 bbl. of cement per cu. yd. were standard-cured for 28 days, then immersed for 6 months in sodium-sulfate solutions under the conditions shown in Table 17. Corresponding cylinders were standard-cured for 28 days, then immersed in water under the same conditions of temperature and exposure. At the age of 7 months, the compressive strengths of the specimens were determined. The results are reported as the ratio of the strength of the sulfate-stored specimens to that of the corresponding water-stored specimens, expressed as percentages.

Considering the specimens immersed in a 1-per cent solution, it is seen in Table 17 that the test results are not greatly different for the two conditions of storage. The medium-lime and low-lime, low-alumina portland cements are more resistant than the high-lime portland cement, as are also the portland-puzzolan cements containing oil-impregnated diatomaceous earth, tuff, basaltic tuff, and waste asbestos rock. Ottawa sand and clay A exhibit a low degree of resistance as measured by this test.

In the 10-per cent solution, the concretes containing the high-lime portland cement and seven of the portland-puzzolan cements disintegrated prior to the date of test (Table 17). The remaining portland-puzzolan cements for which test results are available are considerably more resistant than the medium-lime and low-lime, low-alumina portland cements under these conditions of test.

TABLE 17—RESISTANCE OF PASTES, MORTARS, AND CONCRETES CONTAINING PORTLAND AND PORTLAND-PUZZOLAN CEMENTS TO THE ACTION OF SODIUM SULFATE

Group	Puzzolan[a]	Spec. Surf., sq. cm per g	Neat Slabs: Age at Disintegration in 10 Per Cent Na_2SO_4, Days: Cracking, Faint	Cracking, Distinct	Cracking, Large	Warping, Faint	Warping, Distinct	Warping, Large	Neat Slabs: Index of Resistance[d]	Standard Mortar Briquets[e]: Period of Immersion Before Disintegration, Months: 2 Per Cent Na_2SO_4, Slight	2 Per Cent, Bad	2 Per Cent, Very Bad	2 Per Cent, Complete	10 Per Cent Na_2SO_4, Slight	10 Per Cent, Bad	10 Per Cent, Very Bad	10 Per Cent, Complete	Mortar Bars[f]: Expansion in 10 Per Cent Na_2SO_4, Millionths[g]: 2 Mo.	4 Mo.	Concrete Cylinders[j]: Per Cent of Compressive Strength of Water-stored Concrete (Age 7 Mo.): At 70°F., 1 Per Cent Na_2SO_4	At 70°F., 10 Per Cent Na_2SO_4	Outdoors, 1 Per Cent Na_2SO_4
Diatomaceous Silicas	Diatomaceous earth, calc.	1840	11	15	(b)	15	(b)	(b)	695	2	4	–	–	1	2	3	–	—	—	—	—	—
	Oil-impreg. diat. earth, calc.	1840	11	13	(b)	13	(b)	(b)	691	3	4	–	–	2	3	6	–	150	220	101	72	106
	Diatomaceous shale, calc.	1890	11	11	(b)	15	(b)	(b)	691	3	6	–	–	2	3	5	–	160	230	98	81	95
Volcanic Silicas	Pumicite, calcined	1960	9	9	32	10	83[c]	83[c]	226	2	4	–	–	1	2	3	–	330	420	92	85	95
	Tuff, calcined	1740	11	19	(b)	6	88	(b)	560	3	6	–	–	1	2	3	–	200	200	105	95	116
	Italian pozzuolana, raw	1860	11	19	25	11	29	43	138	2	3	5	–	1	2	3	4	810	(h)	92	0	104
	Basaltic tuff, calcined	1560	6	15	22	8	19	32	102	2	3	4	–	1	1	3	4	650	(h)	105	0	113
Siliceous Rocks	Granite (Arrowrock Dam), raw	1750	9	11	23	9	23	30	105	2	2	3	3	1	1	2	4	(h)	—	96	0	97
	Granite C, raw	1730	6	11	22	11	25	144	219	2	2	3	3	1	1	2	3	—	—	—	—	—
	Ottawa sand, raw	1630	6	8	22	11	19	25	91	2	2	2	3	1	1	2	4	(h)	—	76	0	100
	Waste asbestos rock, raw	1870	13	19	(b)	15	(b)	(b)	701	2	4	–	–	2	3	4	–	160	250	105	103	105
Clays	Clay A, calcined	1730	8	13	19	13	34	43	130	2	2	3	4	1	1	2	2	(h)	—	89	0	92
	Clay B, calcined	1850	11	11	19	7	22	25	95	2	3	5	8	1	1	2	4	820	(h)	92	0	102
	Clay C, calcined	1800	8	11	53	19	144[c]	144[c]	379	2	3	–	–	1	2	4	–	320	700	102	67	97
	Clay D, calcined	1750	11	13	32	13	36	55	160	2	3	4	8	1	1	2	4	1430	(h)	92	0	99
Limestones	Limestone A, raw	1720	14	24	42	18	42	52	192	2	8	–	–	2	2	3	7	—	—	—	—	—
	Limestone B, raw	1640	18	21	49	14	59	61	222	4	8	–	–	2	3	4	6	—	—	—	—	—
Portland Cements	None (high-lime p.c.)	1680	8	11	25	8	19	43	114	2	3	3	5	1	2	3	4	(h)	—	94	0	101
	None (medium-lime p.c.)	1560	18	26	68	18	84	129	343	2	3	4	8	2	2	3	5	180	(h)	100	31	112
	None (low-lime p.c.)	1440	17	18	151	13	27	151	377	2	3	5	–	1	2	3	6	—	—	—	—	—
	None (low-lime, low-alumina p.c.)	1480	10	10	(b)	14	(b)	(b)	688	2	4	–	–	1	2	4	–	270[i]	960[i]	102[i]	58[i]	100[i]

[a]All portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.
[b]No further disintegration observed up to the age of 218 days.
[c]Specimen broke at age shown.
[d]Sum of ages in preceding six columns, with 218 (age of last inspection) in spaces marked "b."
[e]Halves of broken briquets from 28-day tension test; immersed in solution at 28 days.
[f]2 by 2 by 18-in. bars; c/a = 1:4 by wt.; w/c = 0.73 by wt.; standard Ottawa sand; curing 3 days moist at 70°F. before immersion.
[g]Difference between expansion of bars immersed in solution of sodium sulfate and expansion of corresponding bars immersed in water.
[h]Very high expansion; readings discontinued.
[i]Interpolated value.
[j]3 by 6-in. cylinders; c/a = 1:5.65 by wt.; 2½-in. flow in Burmister trough; 0 to ¾-in. San Gabriel gravel; curing 28 days moist at 70°F., then immersion in water or in sulfate solution as indicated.
[k]Specimens partly immersed.

TABLE 18—VOLUME CHANGES AND RESISTANCE TO FREEZING AND THAWING OF MORTARS CONTAINING PORTLAND AND PORTLAND-PUZZOLAN CEMENTS

Group	Puzzolan[a]	Spec. Surf., sq. cm per gram	Length Change of Mortar, millionths[b], [c]					Resistance of Mortar to Freezing and Thawing[e]	
			Water-Cement Ratio, by wt.	Total Expansion in Fog at 70°F.	Net Contraction in Air[d]			Per Cent Disintegration at 100 Cycles	Index[f]
				28 Da.	35 Da.	4 Mo.	1 Yr.		
Diatomaceous Silicas	Diatomaceous earth, calcined	1840	0.59	121	763	1636	1848	—	—
	Oil-impregnated diat. earth, calc.	1840	0.58	121	727	1452	1616	17	701
	Diatomaceous shale, calcined	1890	0.61	112	839	1728	1928	100	494
Volcanic Silicas	Pumicite, calcined	1960	0.52	68	581	1304	1457	100	355
	Tuff, calcined	1740	0.53	112	678	1378	1525	2	700
	Italian pozzuolana, raw	1860	0.56	85	756	1465	1642	83	503
	Basaltic tuff, calcined	1560	0.55	83	620	1311	1469	100	579
Siliceous Rocks	Granite (Arrowrock Dam), raw	1750	0.51	69	617	1255	1399	100	485
	Granite C, raw	1730	0.55	66	756	1371	1524	—	—
	Ottawa sand, raw	1630	0.49	59	583	1179	1305	100	445
	Waste asbestos rock, new	1870	0.63	165	535	1835	2148	100	452
Clays	Clay A, calcined	1730	0.55	91	773	1395	1593	50	619
	Clay B, calcined	1850	0.54	99	628	1258	1409	83	489
	Clay C, calcined	1800	0.54	70	665	1208	1356	75	595
	Clay D, calcined	1750	0.53	68	698	1278	1431	83	484
Limestones	Limestone A, raw	1720	0.53	52	500	1188	1350	—	—
	Limestone B, raw	1640	0.54	52	566	1143	1311	—	—
Portland Cements	None (high-lime p.c.)	1680	0.52	92	583	1158	1236	100	506
	None (medium-lime p.c.)	1560	0.51	34	727	1383	1550	50	642
	None (low-lime p.c.)	1440	0.54	45	681	1508	1756	—	—
	None (low-lime, low-alumina p.c.)	1480	0.57	68	715	1477	1603	30[g]	666[g]

[a]All portland-puzzolan cements contain 20 per cent by weight of puzzolan, blended with high-lime clinker.
[b]1½ by 1½ by 12-in. bars of 1:3.25 mortar containing 0 to No. 4 Niles sand; standard curing to 28 days.
[c]All values referred to length at 2 days.
[d]Preliminary curing in fog at 70°F. for 28 days, then in air of 50 per cent relative humidity at 70°F.
[e]2 by 4-in. cylinders of 1:3 mortar containing 0 to No. 4 San Gabriel sand; fixed consistency; standard-cured 28 days, then (while saturated) subjected alternately to freezing and to thawing at 160°F.
[f]Twice the cumulative number of cycles at which percentage of disintegration is successively 2, 25, 50, and 100 per cent.
[g]Interpolated value.

Comparison of Portland Cements. Considering all of the test results for portland cements in Table 17, it is indicated that (1) the higher the silica content the greater the resistance of portland cements to the action of sodium sulfate; and (2) the lower the alumina content the greater the resistance.

Comparison of Portland-Puzzolan Cements. Considering all of the test results for portland-puzzolan cements in Table 17, it is indicated that on the whole the most resistant portland-puzzolan cements are those containing the diatomaceous silicas, followed in order by those containing volcanic silicas and limestones. As a group, the clays are but slightly more resistant than the siliceous rocks; but clay C and waste asbestos rock (a siliceous rock) are more resistant than the remaining puzzolans in their respective groups.

Based upon the more complete test results, including those for blends with all of the portland-cement clinkers, it appears that all of the portland-puzzolan cements containing the diatomaceous silicas, volcanic silicas, and limestones are more resistant to sulfate action than are the corresponding portland cements.

Effect of Calcination of Puzzolan. The effect of calcination of puzzolan upon the resistance of portland-puzzolan cements to the action of sodium sulfate is shown in part in Table 5. Based upon the few comparisons available from this and the other tests for sulfate resistance, it is indicated that calcination of pumicite lowers somewhat its sulfate resistance; that calcination of diatomaceous earth, diatomaceous shale, and tuff do not greatly affect the resistance; and that calcination of clay improves its resistance.

Effect of Amount of Puzzolan. The effect of amount of puzzolan upon the resistance of portland-puzzolan cements to the action of sodium sulfate is shown in part in Table 8. It is seen that, within the limits of 10 to 30 per cent, the larger the percentage of puzzolan the greater the resistance. Based upon the more complete data, it is indicated that the use of only 10 per cent of puzzolan results in only a slight increase in resistance, and in some cases an actual decrease; but that the use of 20 or 30 per cent of puzzolan considerably improves the resistance over that for the low percentage of puzzolan.

Volume Changes of Mortars

Expansion in Fog. The expansion of standard-cured mortars between the ages of 2 and 28 days is shown in Table 18. The expansion under moist conditions up to the age of 28 days is 1/15 to 1/25 as great as the contraction under drying conditions up to the age of 1 year; and in no case is the expansion great enough to be of significance. The greatest expansion is exhibited by the portland-puzzolan cement containing waste asbestos rock, which is high in magnesia; and the least expansion is exhibited by the portland-puzzolan cements containing the limestones. For 11 of the 17 portland-puzzolan cements, the expansion is less than that for the high-lime portland cement. At the age of 1 year, the values available for a few of the portland-puzzolan cements indicate that their mortar expansion is up to 20 per cent greater than that for the high-lime portland cement.

Contraction in Air. In Table 18 is shown the contraction of mortar standard-cured for 28 days and then stored in air of relative humidity 50 per cent at 70° F. up to the age of 1 year. All values are *net*, being referred to the length at the age of 2 days. Considering the 1-year contraction, it is seen that the contraction is least for the high-lime portland cement and greatest for the portland-puzzolan cement containing the waste asbestos rock. Of the portland-puzzolan cements, the contraction is least for those containing the limestones. Considering the siliceous portland-

puzzolan cements, the contraction is least for the siliceous rocks (except waste asbestos rock), followed in order by the clays, volcanic silicas, and diatomaceous silicas. For only three portland-puzzolan cements (those containing waste asbestos rock, diatomaceous shale, and diatomaceous earth) is the contraction greater than that for the low-lime portland cement.

It has been observed that a consistent relation exists between the gross contraction at later ages and the contraction after relatively short periods of drying. For example, at the age of 4 months (3 months of air storage) the average gross contraction for the cements listed in Table 18 is 0.90 of the contraction at 1 year (11 months of air storage); for the individual cements this ratio ranges from 0.86 to 0.94 and the mean variation from the average is only 0.01. Similarly, for 94 mortars included in this investigation the ratio of gross contraction after only 28 days of air storage is 0.75 of that after 11 months of air storage, with a mean variation from the average ratio of 0.03. Of course these numerical ratios apply only to the particular conditions of testing, which in this investigation are 1:3.25 mortar containing Niles sand, 1½ by 1½ by 12-in. bars, and storage in air of 50 per cent relative humidity at 70° F., but undoubtedly similar ratios exist for any given set of testing conditions. It appears that the relative differences in contraction as between cements develop during a rapid first stage of drying, after which these differences remain in relative proportion during a secondary, slower stage of drying.

Considering those portland-puzzolan cements which contain both raw and calcined puzzolans (Table 5), the net contraction of mortar at 1 year is 7 to 20 per cent less for puzzolan calcined at 1450° F. than for raw puzzolan. For diatomaceous shale calcined at 1800° F., the net contraction of mortar at 1 year is 16 per cent less than that for the same puzzolan calcined at 1450° F.

The effect of amount of puzzolan upon the contraction of mortar in air at the age of 1 year is shown in Table 8. It is seen that the contraction is greater for portland-puzzolan cements than for the high-lime portland cement, and that it is greater for the higher percentages of puzzolan in the blend. However, for the ten puzzolans listed the mortar contraction exhibited by the low-lime cement is not exceeded by the portland-puzzolan cements containing the following amounts of puzzolan:

Percentage of Puzzolan in Portland-Puzzolan Cement	*Puzzolan*
10	Diatomaceous earth, diatomaceous shale
20	Oil-impregnated diatomaceous earth, tuff
30	Pumicite, basaltic tuff, granite (Arrowrock Dam), Ottawa sand, clay A, limestone A

In some cases, approximately 15 per cent of lime (by weight of puzzolanic component) was interground with portland-puzzolan cements containing (1) calcined diatomaceous shale and (2) raw pumicite. Invariably the net contraction of mortar was reduced, in amounts ranging from 8 to 19 per cent, perhaps due to part to the smaller water requirement of the cements containing lime. The contraction was reduced more by the use of hydrated lime than by the use of quicklime.

Fig. 2 shows for three aggregates and three cements the effect of type of aggregate upon volume changes of mortar. It is seen that at the age of 4 months (1) the contraction of mortar containing Boulder Dam sand (a quartz sand containing some limestone) is about two-thirds of that for Niles sand (a siliceous sand containing considerable amounts of disintegrated sandstone), and (2) the contraction of mortar containing limestone screenings is only one half of that for Niles sand. When the limestone aggregate is employed the mortars containing the portland-puzzolan

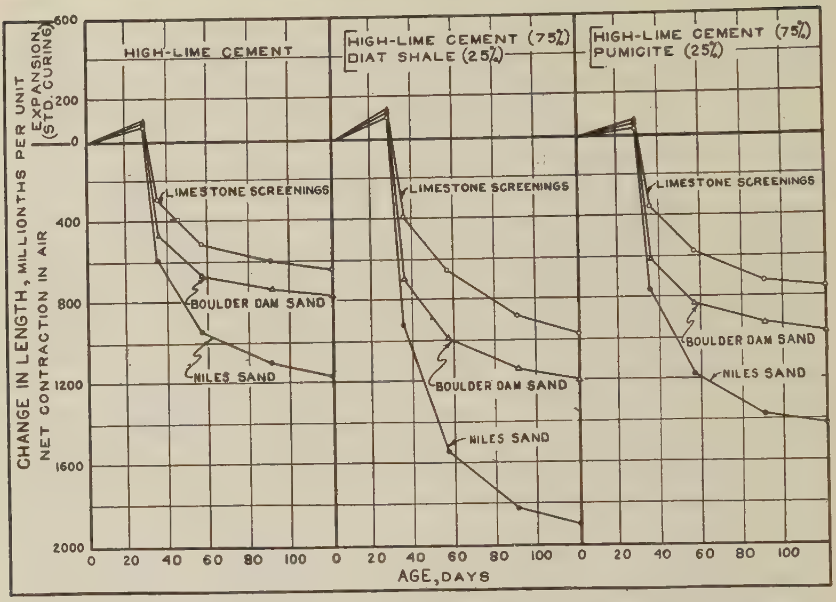

FIG. 2—EFFECT OF TYPE OF AGGREGATE UPON VOLUME CHANGES OF MORTAR

cements exhibit but little greater contraction than that for the high-lime portland cement. Further, the contraction of the portland-puzzolan-cement mortars containing limestone screenings is less than that of the portland-cement mortar containing Niles sand.

Attention is called to the relatively low contraction of mortars containing lime in any of the three forms employed in this investigation—as limestone dust interground with the cement, as quicklime or hydrated lime interground with the portland-puzzolan cements, or as limestone aggregate.

Thermal Expansion. A few tests were made to determine the relative coefficients of thermal expansion of moist mortar bars similar to those employed in the expansion tests but cured in the molds at 70° F. for 2 days, then in sealed containers at 100° F. for 1 day, then in the sealed containers at 130° F. up to the age of 28 days. Within the range of 130 to 70° F., the coefficient of thermal expansion at 28 days is as follows, expressed as a percentage of that for the high-lime portland cement:

Portland-Puzzolan Cement Containing 30 Per cent of Puzzolan Listed Below	*Percentage of Coefficient for High-Lime Portland Cement*
Limestone A, raw	88
Ottawa sand, raw	92
Tuff, calcined	103
Pumicite, raw	107
Shale, calcined	124

Resistance of Concretes to Freezing and Thawing

In Table 18 are shown the percentages of disintegration of concrete specimens standard-cured for 28 days and then (while continuously saturated) subjected to 100

alternations of freezing and of thawing at 160° F. A further indication of the relative resistance of the cements is given by the corresponding "index" values. Considering these values, it is seen that (1) of the portland cements, the high-lime cement is least resistant; and (2) of the portland-puzzolan cements, the following exhibit greater resistance than that of the high-lime portland cement, ranging from high to low resistance in the order named; oil-impregnated diatomaceous earth, tuff, clay A, clay C, and basaltic tuff. The lowest resistance to freezing and thawing is exhibited by the portland-puzzolan cements containing pumicite, Ottawa sand, and waste asbestos rock.

For only one puzzolan (pumicite) are data available to show the effect of calcination upon resistance to freezing and thawing. It will be recalled (Table 5) that calcination of this particular puzzolan decreases its activity and sulfate resistance. With respect to freezing and thawing also, the calcined pumicite is less resistant than the raw pumicite, having an index value about ⅓ less than that of the raw pumicite. This relation does not necessarily hold true for puzzolans whose activity is increased by calcination.

For four puzzolans (calcined diatomaceous shale, raw pumicite, calcined basaltic tuff, and calcined clay A), tests were made on portland-puzzolan cements containing various amounts of puzzolan (10 to 30 per cent). For all of these puzzolans, the resistance to freezing and thawing is less for larger percentages of puzzolan, but the portland-puzzolan cements containing 20 per cent of puzzolan are equally resistant as, or more resistant than, the high-lime portland cement.

CONCLUSIONS

The applicability of these conclusions is limited to concretes under conditions similar to those of this investigation with regard to type and proportions of materials, fineness of cements, cement content and water-cement ratio of concretes, and curing conditions.

In any comparison of the merits of portland-puzzolan and portland cements, the type of structure for which the concrete is intended must be considered. For mass-concrete structures, it is desirable that the concrete be low in heat generation and (to resist cracking) strong in tension. For hydraulic structures, it is desirable that the concrete be resistant to percolation of water. For structures exposed to an aggressive water, it is desirable that the concrete be resistant to percolation and resistant to corrosion. For structures subjected to continued drying conditions, low contraction upon drying is desirable.

1. Of the four types of portland-cement clinker herein considered, on the whole it appears that the use of a high-lime clinker in portland-puzzolan cements results in the greatest relative benefits as compared with the corresponding portland cement (Table 7).

The following conclusions are drawn principally with respect to portland-pozzolan cements containing high-lime portland cement.

2. In portland-puzzolan cements, the diatomaceous silicas as a group exhibit the highest degree of grindability, strength, and resist-

ance to sodium sulfate; but this group also exhibits large contraction of mortar upon drying (Table 4).

3. The volcanic silicas as a group also exhibit high grindability, strength, and sulfate resistance; and the contraction of mortar upon drying is considerably less than that for the diatomaceous silicas. Of this group, basaltic tuff is relatively low in grindability and sulfate resistance (Table 4).

4. The clays are near the average of the five groups of puzzolan with respect to grindability, strength, and contraction upon drying; but are low in resistance to sodium sulfate. Clay C is markedly better than the other clays (Table 4).

5. The limestones exhibit low grindability and strength, but fairly good resistance to sodium sulfate and the highest degree of volume-constancy (expansion under moist conditions, contraction in air, and thermal expansion) (Table 4).

6. The siliceous rocks as a group exhibit low grindability, strength, and sulfate resistance, but low contraction upon drying. In this group, waste asbestos rock is exceptional in that it exhibits the highest sulfate resistance and highest contraction of mortar upon drying of all puzzolans (Table 4).

7. Of the various tests proposed to determine the activity of a puzzolan, or its ability to combine with lime, the compression test on puzzolan-lime-sand mortar cured at 130° F. between the ages of 1 and 28 days provides a fair indication of the contribution of the puzzolan to the strength at later ages of mortar containing the portland-puzzolan cement (Table 3).

8. The highest degree of activity is exhibited by the diatomaceous silicas, followed in order by the volcanic silicas, clays, siliceous rocks, and limestones (Table 1).

9. No definite relation is apparent between the chemical composition of puzzolan and the properties of portland-puzzolan cements in mortars and concretes; but in general as between active puzzolans it appears that the higher the silica content the higher the strength and sulfate resistance of the portland-puzzolan cements in mortars and concretes (Tables 1 and 4).

10. In general, the lower the specific gravity of puzzolan, the better the strength and sulfate resistance of the corresponding portland-puzzolan cement in mortars and concretes. However, small differences in specific gravity do not appear to be significant.

11. For portland-puzzolan cements containing each of six puzzolans blended both in the raw and in the calcined state, calcination of the puzzolan at 1450° F. affects but little the grindability of cement and

the tensile strength of mortar, but decreases considerably the contraction of mortar upon drying. Calcination of the puzzolan increases greatly the sulfate resistance for diatomaceous earth and clay, but lowers considerably the resistance for pumicite and tuff (Table 5).

12. The optimum percentage of a particular puzzolan in portland-puzzolan cement necessarily depends upon the property of mortar or concrete under consideration, as well as upon the type of clinker. In general, increasing the amount of puzzolan (up to 30 per cent) improves the grindability and increases the resistance to sodium sulfate, but decreases the strength and volume-constancy. On the whole, consideration of the foregoing properties indicates that the optimum amount of active puzzolans when blended with high-lime clinker is likely to lie within the limits 10 to 30 per cent (Table 8).

13. With few exceptions, the portland-puzzolan cements are of greater fineness (more grindable) than the portland cements with which the puzzolans are blended, when ground under the same conditions for equal periods of time (Table 9 and 10).

14. In general, the water requirement for fixed consistency of neat paste, mortar, or concrete is greater for portland-puzzolan cements than for the portland cements with which the puzzolans are blended. The greater the percentage of puzzolan, the greater the water requirement (Tables 9 and 11).

15. It has been observed that, in general, portland-puzzolan cements in concrete exhibit greater water-retaining capacity and greater resistance to percolation than the portland cements with which the puzzolans are blended (Table 12).

16. In standard mortar, at the age of 3 months the groups of portland-puzzolan cements containing 20 per cent of diatomaceous silicas, volcanic silicas, and clays, blended with high-lime clinker, exhibit higher tensile strengths than that for the high-lime portland cement; and the group containing diatomaceous silicas also exhibits higher compressive strength (Table 14).

17. Under conditions of air storage after moist-curing for 28 days, at the age of 3 months the tensile strength of standard mortar is higher for practically all of the portland-puzzolan cements blended with high-lime clinker than for the portland cements (Table 13).

18. In relatively rich concrete, containing 1.5 bbl. of cement per cu. yd., at ages of 3 months and 1 year the portland-puzzolan cements exhibit relatively low compressive strengths as compared with the portland cements (Tables 13 and 14). In leaner mixes (1 bbl. or less of cement per cu. yd.), at the age of 3 months the compressive strengths

exhibited by the portland-puzzolan cements are in general higher than the strengths for corresponding portland cements (Table 15).

19. Of the portland-puzzolan cements, at the age of 1 year the group containing volcanic silicas exhibits the highest compressive strength of concrete (Table 14).

20. Part of any beneficial effect of a puzzolan may be that of a void filler, increasing the workability and density of the concrete, especially in lean or harsh mixes. Even the inert materials employed in portland-puzzolan cements produce higher compressive strengths of concrete than the corresponding portland cement reduced to the amount of portland cement in the portland-puzzolan cement (Tables 13 and 14). It has been observed that, for equal consistencies, concretes containing portland-puzzolan cements are more plastic than concretes containing portland cements.

21. The range in modulus of elasticity of concrete is not great as between the various portland-puzzolan cements. On the average the modulus is about nine-tenths of that of the corresponding portland cement (Tables 13 and 14).

22. No pronounced difference in absorption is apparent between concretes containing the various portland and portland-puzzolan cements. For the concrete employed, the average value of absorption is about 7 per cent (Table 13).

23. The unit weight of the concretes containing portland-puzzolan cements is on the average about 3 lb. per cu. ft., or about 2 per cent, less than that for the corresponding portland cements (Table 13).

24. The portland-puzzolan cements generate less heat than the portland cements with which the puzzolans are blended, the reduction in heat generation being roughly proportional to the percentage of puzzolan. A larger proportion of the heat is generated at the early ages for portland-puzzolan cements than for portland cements (Fig. 1).

25. Considering portland-puzzolan cements containing 20 per cent of active puzzolan blended with high-lime clinker, at the age of 28 days the ratio of tensile strength of standard mortar to the heat of hydration of cement is greater than that for the high-lime portland cement, and is equal to or greater than that for the low-lime portland cement (Table 16).

26. The portland-puzzolan cements containing diatomaceous silicas, volcanic silicas, and limestones are more resistant to the action of sodium sulfate than are the corresponding portland cements, provided that the puzzolans are present in amounts greater than 10 per cent and up to 30 per cent, the limit of these tests (Tables 17 and 8).

27. On the whole, as between portland-puzzolan cements the highest resistance to the action of sodium sulfate is exhibited by those containing the diatomaceous silicas, followed in order by those containing volcanic silicas, limestones, clays, and siliceous rocks. Clay C and waste asbestos rock (a siliceous rock) are more resistant than the remaining puzzolans in their respective groups (Table 17).

28. The contraction of mortar upon drying is greater for the portland-puzzolan cements than that for the corresponding portland cements; but for only three of the portland-puzzolan cements containing 20 per cent of puzzolan blended with high-lime clinker is the contraction greater than that for the low-lime portland cement Table 18).

29. The mortars containing lime, whether in the form of limestone dust interground with the portland cement, in the form of quicklime or hydrated lime added separately to the portland-puzzolan cement during manufacture, or in the form of limestone aggregate, exhibit considerably less contraction upon drying than the mortars containing siliceous materials (Table 18, text, Fig. 2). Portland-puzzolan cements containing limestone also exhibit low expansion with time under moist conditions and low thermal expansion.

30. Under various conditions of testing in all of which 1½ by 1½ by 12-in. mortar bars are exposed to drying in air of 50-per cent relative humidity at 70° F., for a wide variety of cements a consistent relation exists between the gross contraction at later ages and that after relatively short periods of drying. For fixed conditions of test, it appears that such a relation may be established and thereafter employed to predict the ultimate contraction from short-time tests.

31. Higher resistance of concrete to the action of freezing and thawing is exhibited by five portland-puzzolan cements containing 20 per cent of puzzolan blended with the high-lime clinker than that exhibited by the high-lime portland cement. Of these five portland-puzzolan cements, the highest resistance is exhibited by oil-impregnated diatomaceous earth, followed in order by tuff, clay A, clay C, and basaltic tuff (Table 18).

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*See selected references in Institute paper referred to on page 82.

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3. Grun, R.: "Cement with Pozzolanic Admixtures," Proc. International Assn. Testg. Mat., Vol. 1, 1931; translated by U. S. Bur. Rec. in Tech. Memo. 419, Denver, 1934.

4. Menzel, Carl A.: "Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete," JOURNAL Amer. Concrete Inst., November-December 1934 (Proc. Vol. 31), pp. 125-148. (Shows effect of finely divided silica upon properties of mortar and concrete).

For such discussion of this paper as may develop readers are referred to the JOURNAL *for March-April 1936. Discussion should reach the Secretary by February 1, 1936.*

Discussion of a paper by P. H. Bates:

"Trends in the Production and Use of Various Types of Hydraulic Cements"*

CONVENTION DISCUSSION

Robert W. Lesley (*Philadelphia*): I do not want to get into the discussion, but I do want to pay a compliment to the very interesting paper we have heard today. It sets one of us antiques to thinking. Having been associated with and been personally one of the earliest manufacturers of cement in this country, I believe from the beginning that portland cement manufacturers had something more than money to go for; they had a reputation to make for a new commodity in this country. When I started in manufacturing cement, I think the total output was about 5,000 barrels, and in my short lifetime I have seen it grow to 175 million barrels; certainly the baby born many years ago must have lived a decent and respectable life. Now Mr. Bates gives us a 98 per cent perfection.

Taking that figure and the fact that, as near as I can gather from random figures—some of them before my time—we have probably used more than a billion barrels of cement in every kind of work, under all conditions of weather, all conditions of sand and stone and human labor—yet our speaker gives us a ninety-eight per cent perfection.† Who of us would not like to go to a college or school and come out with a diploma like that? I call it a mighty fine record, and when I talk to Mr. Bates he is frank enough to say, which most people have been forced to say, that the fault was not with the cement but generally with other elements that enter into the use of cement.

When we think that in the construction of a building such as this, nearly everything is brought to it in a state of completion—if these columns are steel or iron, they are brought here as a finished product, but if they are concrete, you bring the cement here as one element, the sand as another, the water as another element and the rock as another element, and then throw human labor into their midst.

*Presented at the 31st annual convention, New York, Feb. 19-21, 1935. Journal, Amer. Concrete Inst., Jan.-Feb., 1935; *Proceedings* Vol. 31, p. 225.

†Mr. Bates said: "granted that 98 per cent of all installations are eminently satisfactory, it must be acknowledged that 2 per cent or less of failures can cause many heartaches."

That is a very serious combination when you manufacture columns and girders and beams to carry a building ten stories high, five stories high or three stories high, and manufacture the carrying members on the site subject to all the elements of weather, labor and every other condition. So, while I may be old fashioned, may be dug out from the ruins, but still I do say here, and I am sure Mr. Bates will agree with me, that we have made a pretty darned good showing for the material which you gentlemen are all interested in. I was the first chairman of the Committee on Concrete and Reinforced Concrete and I think I was one of the first gentlemen who formed this Society; I know I was a charter member of the American Society for Testing Materials, and maybe I was fool enough, having a brand of cement of our own that we could get a quarter of a dollar more for, to be the first man to preside over C-1, the committee that makes the specifications for cement. I was putting my own self out of profitable business to get something that would be standard in this country. So when Mr. Bates speaks of the good old standard cement I am with him to the last point. When we come to the "57" varieties of cement, more or less, all I can say is God help the cement manufacturers and the cement consumers and the reputation of a well established product made carefully, made with honor and sold upon honor to the consumer at large. I am against all the modern methods and the new untried tests that have been devised and on what test of time is not known. Proposers talk not only of the materials that enter into cement by the ordinary mechanical processes, but the proportional compounds in a barrel of cement. The gentleman charged with the examination of the 2,000 cement laboratories in the United States, was frank enough to tell us that he had examined a thousand of them, and another thousand he had not got to yet and many of the first thousand he found defective. If we come to complicating our tests and our specifications, we must have the laboratory perfect; all the material in the laboratory must be perfect; the chemicals used in making all these new analyses must be in absolutely perfect condition, certified by the manufacturers; and then we have the perfection of test methods and machines; last of all it might be well to have a test, the old army test, for human intelligence, of the men in the laboratories. It seems going a little far, but while we are testing everything, let us test them, and then we will have some kind of record that we can go to bat with. I still stand on the feeling that, after more than 50 years of manufacturing and a billion barrels of cement with a failure of two per cent which is not yet traced absolutely to the cement, that the good old cement can stand the racket. We will make any darned thing anybody wants, if he is willing to take the risk.

Discussion of a paper by Benjamin Wilk:

"High Early Strength Cements in Concrete Products Manufacture"*

CONVENTION DISCUSSION

J. C. Pearson (*Director of Research, Lehigh Portland Cement Co., Allentown, Pa.*): This difference Mr. Wilk has pointed out in the behavior of high early strength cement in sustaining its efficiency at alter periods, is interesting. As a first consideration, I do not doubt that the early cessation of curing is one of the reasons for that sustained difference. However I think there is another factor which involves the fineness, and perhaps focuses our attention on that special phase of high early strength cement. I know in the old days we had experiments in which two hundred mesh material was sieved out of the cement and the coarse residue made up into neat or mortar briquettes, and it was found that the neat briquettes developed very little strength, and the mortar briquettes almost none at all. From this it was concluded that the coarse particles had no cementing value, and that as much as fifty per cent of the cement might be regarded as inert aggregate. We know now that those clinker particles are far from being inert, but when you have a large area of aggregate to coat, as in these block mixtures, it is quite obvious that the coarse particles have little value. If they were ground up into fine cement, then they would contribute more of the real paste which does the gluing act. To show how much more that can be, even when the quantity of high early strength cement is reduced, I have prepared for my own amusement a couple of tubes here which show the regular cement and the high early strength cement, each of them separated into different sized particles by means of air separation, and then packed into the tubes in the same proportion in which they occur in the two products. You can see from the different colors how much the so-called flour of the cement is increased. These happen to be two cements made at one of our plants in which the difference in fineness is the striking thing. The

*Presented at the 31st annual convention, American Concrete Institute, New York, Feb. 19-21, 1935. Journal, Amer. Concrete Inst., Jan.-Feb., 1935; *Proceedings* Vol. 31, p. 241.

point is that the quantity of the flour in the high early strength cement is much greater, so that even if you reduce the total amount of the early strength cement by twenty-five per cent, as Mr. Wilk did, in order to get the two cements on the same cost basis, you still have more flour and therefore more of the kind of paste which is really effective in these lean mixtures. Thus the compressive strengths given by Mr. Wilk show an actual efficiency (that is, strength per pound of cement) for the early strength cement about 60 per cent higher than that of the regular cement, whereas its cost, as just stated, is only 25 per cent to 30 per cent higher.

(*To be concluded in November-December* JOURNAL)

Discussion of a paper by Inge Lyse:

"Effect of Brand and Type of Cement on Strength and Durability of Concrete"*

CONVENTION DISCUSSION

D. L. Snader (Stevens Institute of Technology, Hoboken, N. J.): I should like to ask Professor Lyse whether he means to indicate that freezing and thawing are the only factors involved in durability?

Professor Lyse: No; with the durability index we refer only to this particular type of test, that is freezing and thawing. In structures subjected to other durability exposures, we ought to have other indexes.

President Bates: Professor Lyse, some of your conclusions interest me extremely. Will you read your No. 15?

Professor Lyse: "Variation in strength and durability of these different cements was probably more due to the method of manufacture of the cement than to any other cause."

President Bates: Do you include composition among other causes?

Professor Lyse: Yes sir.

President Bates: I am glad to have you say that. I think we will realize more and more that we are going to improve portland cement more by manipulation in its manufacture than by its compounds. We are getting lots of proof of that; it is something for the manufacturer to begin to study.

Professor C. H. Scholer (Kansas State College, Manhattan, Kan.): I think it is for some eight years that I have been testing concrete by freezing and thawing. I think in two or three more years I won't dare to say any more about the variables connected with cement. There are a lot of variables in the freezing and thawing test. In general, the trends follow Professor Lyse's claim—as the cement content increases the durability increases; it follows the water-cement ratio law as does the strength relationship. Yet, if you want to pick out by the freezing and thawing test, which brand of cement to use, then run only one test. If you run two tests, you won't know so much about it, and by the time you have run three or

*Presented at the 31st annual convention, American Concrete Institute, New York, Feb. 19-21, 1935. Journal Amer. Concrete Inst., Jan.-Feb. 1935; *Proceedings*, Vol. 31, p. 247.

four tests you will almost give it up. We found on some of our tests a very remarkable and outstanding durability indicated by a certain brand of high early strength cement. We duplicated those tests with what we thought was better control than before, and revised our judgment considerably. There had been some change made in the manufacture of that cement in the meantime. Our results generally indicate that all the high early strength cements show remarkably high durability on freezing and thawing tests, and our results with high alumina cement have in general been very outstanding, probably more so than with any other cement. Among the lower strength cements variations in the freezing and thawing test are greater. The degree of exposure, the rate at which the heat is transferred, the thawing temperature and probably the critical temperature range which you get from the low end of the freeze to the high point of the thaw vary the severity of the test. We have taken specimens, put them in water, and run the water through the same temperature changes without freezing, and broken the concrete down from that condition alone. With such a test procedure the aggregate used will make a lot of difference. Before drawing conclusions about the kind of cement to use, select your aggregate, because it is probably going to have a larger effect than your cement—we have some very bad aggregates in Kansas, almost as bad as in some other states. Probably the treatment of the cement in manufacture has an important effect as Professor Lyse observed. Recently several outstanding contributions to the literature of freezing and thawing have come from Germany. I have had several of those translated. They are intensely interesting. A. S. T. M. Committee C-9 has some of them.

President Bates: May I ask Professor Scholer if we must select durable aggregate also and run only one test?

Professor Scholer: No, that is simpler. The product from most aggregate sources is so variable that you must run tests frequently.

Professor Lyse: We took every precaution we could think of to insure uniformity. We had the same water content, we used the same type of aggregate for all the tests and had all the specimens submerged completely; we also had them frozen and thawed on the same day; that is, they all went through the same cycle of freezing and thawing under identical conditions, so that variations in the test procedure were thus reduced to a minimum. The only thing we necessarily had to vary was the ratio between the cement and aggregate as we went from a lean to a rich mix.

(*To be concluded in November-December* JOURNAL)

Discussion of a paper by Hardy Cross:

"WHY CONTINUOUS FRAMES"*

BY ARTHUR G. HAYDEN†

I am interested in the statement by Prof. Hardy Cross at the convention that hinged articulation just above the footing of a concrete rigid frame bridge is preferable to continuous construction from base of footing to base of footing.

I share the general feeling among engineers that Professor Cross is one of the most practical as well as one of the profoundest thinkers in the field of structural analysis. Thus, his statements will be accepted almost without question and for that reason I feel impelled to state why I differ from him upon the point of articulated hinges in rigid frame bridges, such as we have developed in Westchester and such as were under discussion.

In the design of our concrete frame bridges we recognized that without hinged articulation above the footing there was some uncertainty about the stresses due to the fact that it is impossible to locate precisely the point on the base through which the resultant reaction will pass. It is certain, however, that unless the footing can be fixed at the base so that rotation is effectively prevented (which would be impractical construction) the resultant reaction cannot by any possible means pass outside the base. Unless we posit infinite soil pressure resistance, the reaction must come some distance in from the edge of the base. Stresses may therefore be analyzed for extreme conditions as for reactions falling between the edges of the base and we may be sure that such stresses can never be exceeded. If we proportion the structure for these "outside" stresses and find that the economic loss in materials does not exceed the cost of hinged articulation, there can be no advantage in providing physical hinges. We have as a matter of fact found that, up to 80-ft. span at least, hinges are unnecessary and that construction may be simplified by omitting them.

In the design of our earlier structures we analyzed the stresses for

*Presented at the 31st annual convention, American Concrete Institute, New York, Feb. 19-21, 1935. JOURNAL, Amer. Concrete Inst., Mar.-Apr., 1935; *Proceedings* Vol. 31, p. 358.

†Designing Engineer, Westchester County Park Commission, White Plains, N. Y.

the worst possible conditions until we were convinced that such precision was unnecessary and that an analysis based upon the assumption that the reaction point was near the middle of the base for all sections was as accurate as need be. There is no doubt in my mind, therefore, that articulation is purely an academic desideratum and has no place in practical construction of short span frame bridges.

(*To be concluded in November-December* JOURNAL)

Discussion of a paper by B. Moreell:

"Articulations for Concrete Structures—the Mesnager Hinge"*

CONVENTION DISCUSSION

Chairman Alfred E. Lindau (Chicago): I am sure that you have enjoyed this paper. Perhaps some of your prejudices, if any have accumulated through the years, against this type of hinges or articulation, may have been removed by Commander Moreell's exposition of this work. Are there any questions you want to ask him? Well, I want to ask one: Was it important, or apparently important, from your tests that the stirrups or the lateral reinforcement be in intimate contact with the main bars? Was it worth while welding them at that point? Suppose they were placed carelessly and away from the bar, and some concrete or something got in between, might that cause failures, secondary failures, in the concrete?

B. Moreell: If you refer to the tests of the hinged blocks made at the Bureau of Standards, I think it was very important that the lateral reinforcement be in intimate contact with the hinge bars themselves, because we found that a variation in location and in the amount of the lateral reinforcement affected materially the strength of the hinge; in other words, the critical condition there appeared to be the lateral distension of the block, and that was prevented by effective lateral reinforcement. If you refer to the lateral reinforcement in the tests of the arch at the Washington Navy Yard, that arch is not an economical arch and Professor Cross made a statement which explains the reason. It is not an economical arch because the architect would not let us make it that. We wanted to use something that approached more nearly a parabolic arch, but they said that was not beautiful, so we designed an arch, whose center two-thirds is parabolic, and then went into a two-center arch, so that the dead load funicular polygon deviates considerably from the axis of the arch. When we consider the possibility of buckling of the hinge steel in that arch, it is not very great. The critical stress is on the tension side of the arch rib, and that is where failure occurred.

*Presented at the 31st annual convention, American Concrete Institute, New York, Feb. 19-21, 1935. JOURNAL, Amer. Concrete Inst., Mar.-Apr. 1935; *Proceedings* Vol. 31, p. 368.

F. E. Richart (University of Illinois): To pursue that point a little further—I notice that three of the test hinges described showed cracks. I wonder if that was due to the lack of adequate lateral restraint by the bars? While I am on my feet I will ask another question. To use Mr. Moreell's words last year, "I may be sticking my neck out," but since we don't usually have the playful custom they have over in Europe, I will try it. The angle of rotation used in these tests was something like two-hundredths of a radiant. When I first looked at that it seemed an enormously large rotation in a rigid frame bridge. If one wanted to use hinges of this kind, he would run into rotations of thousandths of a radiant rather than hundredths. I wanted to ask if those were much larger than would be had in practice, or whether, because of this flexible type of structure, they were of the nature you would have in service?

B. Moreell: In reply to Professor Richart's first question, he supplied the answer himself by his tests on reinforced concrete columns. In other words, the stress condition in the hinge block is similar to that in spirally reinforced concrete columns—you can make the column very flexible and very strong by adding additional lateral steel. Apparently we did not place enough lateral steel in these blocks to place the critical point in the hinge opening; that is, when the bars at the hinge opening were covered with mortar, the critical point was in the block, and the strength was dependent upon the lateral steel.

Now in regard to the magnitude of the rotation; Mesnager stated that a rotation of 0.0065 of a radiant would be rarely encountered. He calculated that as the probable maximum, but he was thinking about bridges and he was thinking about bridges with parabolic ribs. That is probably true for that kind of structure, but when we go to a structure such as we had here, we do not have a rib which conforms to the load thrust lines—we get larger rotations, and, as I recall it, in the actual calculation of stresses in this arch, we found a maximum rotation at the crown hinge of a little more than 0.02 of a radiant under our design loads. We tried to assume conditions which would take care of a maximum variation of probable conditions, and so in designing this arch we analyzed the stresses for various values of the modulus of elasticity of concrete, assuming that under the dead load it might vary anywhere from 10 to 40, and under the live load anywhere from 10 to 20, and we calculated the stresses in the rib and also the rotations of the hinges under all of these conditions to be sure that we had them all covered.

(*To be concluded in November-December* JOURNAL)

Discussion of a paper by Messrs. Ruettgers, Vidal and Wing:

"An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam"*

BY M. MARY†

COMPARISON OF THE OBSERVED PHENOMENA

Concrete permeability tests have been made at the Ponts et Chaussees Laboratories and two articles have already been published in the *Annales des Ponts et Chaussees* (March-April 1933 and November-December 1934‡ issues). For these tests, the general procedure did not differ materially from that used by the Bureau of Reclamation. However, the inflow of water was not measured; only the outflow was recorded. A few observations are suggested by the comparison of the results:

In general the concretes tested by the Ponts et Chaussees Laboratories were much more permeable than those used by the Bureau of Reclamation.

In most of the tests water appeared on the underside of the specimens immediately after the pressure was applied and the rate of flow decreased rapidly as indicated by the curve shown in Fig. 1. We also obtained results approximating the trend of the curve shown in Fig. 2, which corresponds to less permeable concrete, but even for the latter the rate of flow was greater than recorded by the Bureau of Reclamation. For the most impermeable specimens the coefficient $K_c \times 10^{12}$ according to the definition given by Messrs. Ruettgers, Vidal and Wing, did not drop much below 500 (American units) and for the more permeable ones it reached 50,000.

A comparison of the concretes used in each laboratory does not explain these differences.

*JOURNAL, Amer. Concrete Inst., Mar.-Apr. 1935; *Proceedings* Vol. 31, p. 382.

†Engineer at Ponts et Chaussees (Department of Roads and Bridges) France.

‡Reviewed by B. MOREELL, JOURNAL Amer. Concrete Inst., May-June 1935; *Proceedings* Vol. 31, p. 571.

The factors appearing to control the permeability of concrete are the following:

(a) *The Quantity of Cement*—It seems to be the same magnitude in both cases; 10 per cent to 12 per cent by total weight of the dry materials.

(b) *The Grading of the Aggregates*—Our tests indicated that only the grading of the particles smaller than 0.5 mm. (passing a No. 28 sieve) influenced the permeability, and even then the amount of very fine elements in the mixture must be considerable (from 4 per cent to 5 per cent of the total weight of dry materials passing a 300 sieve) in order to affect the permeability appreciably. It does not appear that the concretes used by the Bureau of Reclamation were studied in this connection.

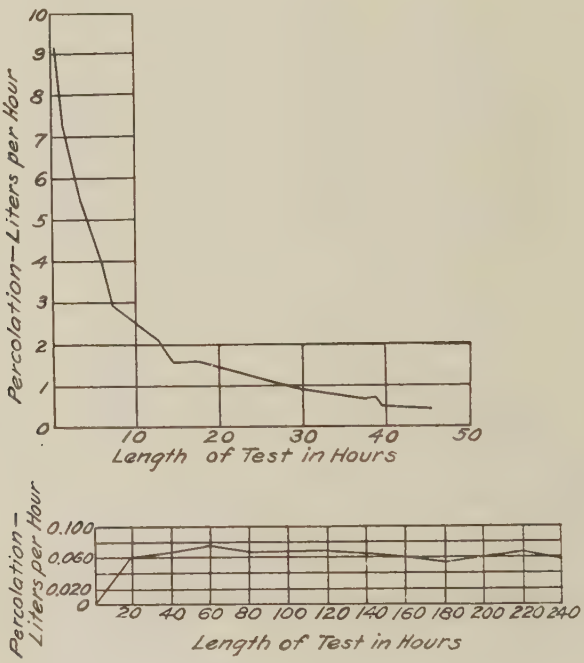

FIG. 1 AND 2

(c) *The Maximum Size of the Coarse Aggregate*—The maximum size of the aggregate used at the Ponts et Chaussees Laboratories was 50 mm. The results therefore should lie in the midst of those obtained by the Bureau of Reclamation.

(d) *The Age of the Specimens*—Little difference exists between the specimens of the two laboratories.

(e) *Method of Curing*—Some of the specimens of the Ponts et Chaussees Laboratories were cured in air, some under water and some at the variable temperature and moisture content of the atmosphere.

Curing under water, because of the swelling of cement, gives better results than air curing. Specimens cured in air and tested with distilled water gave a rapidly decreasing rate of flow as shown in Fig. 1. This phenomenon indicates that the percolation of water through concrete rapidly reduces the flow.

The method of curing used by the Bureau of Reclamation seems to hold a middle course between air and water curing. However, our specimens cured under water have a higher coefficient of permeability than those of the Bureau of Reclamation.

(f) *The Water-Cement Ratio*—According to the tests of the Bureau of Reclamation, this ratio has a considerable influence on permeability. Laboratory specimens of very dry concrete are much less permeable than for the wet mixes. Comment on these results and their practical application will be delayed.

Our tests have given similar results although they were limited to water-cement ratios between .70 and 1.0, as it seems to us impossible to use drier mixtures without obtaining an unsuitable consistency, except of course for mixtures very rich in cement.

The several differences outlined above, particularly the very low water cement ratios used by the Bureau of Reclamation explain partially the difference in the results obtained.

There is still an important gap which we have not been able to explain. It is possible that this difference is due to differences in the characteristics of the cements used. Comparative tests in this respect would be very instructive and would no doubt be a valuable contribution to the Sub-committee on Special Cements of the International Commission on High Dams.

The experiments conducted at both laboratories are in complete agreement with regard to dissolution of cement by percolating water; the phenomena observed are the same and the lime concentrations are closely comparable.

REMARKS ON THE GENERALIZATIONS AND CONCLUSIONS OF RUETTGERS, VIDAL AND WING

(1) The authors state (page 398) that one of the principal reasons for making permeability tests is the study of the decomposition of cement by percolating water. I am in full accord with their views; I even believe it is the only reason, for permeability tests of concrete do not seem to have much bearing on the water-tightness of structures.

Percolation through concrete structures is of two kinds; that which passes through large openings (honeycomb, various imperfections of placement, cracks) which are by far the most important, and that which follows the capillary channels inherent in concrete generally conceived as homogeneous and dense and which characterizes and defines the technical term "permeability of concrete."

Considered in this limited sense, the permeability of concrete plays but a secondary role in the general problem of making structures water tight if one considers merely the total amount of water given passage. The study of this property is of importance only if one considers with it the progressive destruction of concrete by percolating water and the outside agents. As slow as it might be, percolating water dissolves considerable quantities of lime.

(2) Can the coefficient of permeability K_c serve as a means for serious comparison of results obtained by different investigators? Is it sufficient to apply a corrective factor to the readings to take account of end conditions? It seems very doubtful to us. Tests indicate that percolating water modifies very seriously the physical and chemical structure of concrete; water swells the cement particles and renders it less permeable; on the other hand it dissolves a part of the free lime of the cement and causes a transfer of lime to the interior of the mass, and in addition, if the water is calcareous, even to a light degree, it forms on the surface of the test specimen a thin impervious shell of carbonate of lime. The specimen consists then of a series of layers very dissimilar in their characteristics and possessing unequal permeability.

Depending on the test procedure used and the characteristics of the specimen, there seems to be no reason to expect that correction coefficients will actually make the specimens comparable. All the more reason why the coefficient of permeability obtained by tests does not appear to us as being applicable to actual structures.

(3) The trend of curves of percolation show that long time tests are necessary to fix the coefficient of permeability. Under such conditions, is it possible to establish the age of the structure which corresponds to that of the test specimen used in determining K? It is not likely.

Under the circumstances, it seems to us that when two concretes are compared for percolation, it is better to compare the curves in their entirety rather than to limit the comparison to a single arbitrary point, corresponding to a time when the concrete has been materially altered by the passage of water.

(4) The paper lays great stress on the influence of the water cement ratio. All laboratories are in accord on this point, but unfortunately their results are not in agreement with those obtained in the field.

Current observations show that structures in which the concrete was of high fluidity are much less pervious than those in which dry concrete was used even when thoroughly tamped or vibrated.

This result is attributable first of all to the inevitable shortcomings of the methods of tamping. It is true that the paste might be better compacted and less permeable in places where it is properly tamped but the carelessness of a workman, a bad joint or a nest of large stones is all that is needed to create a leak much more important than the percolation resulting from the use of a slight excess of mixing water.

In fact, for massive structures, we are of the opinion that reasonable excess of mixing water can have a beneficial influence on the quality of the concrete since the hardening process takes place in the midst of a damp medium which results in a slight increase in strength and a considerable lowering of the permeability. Therefore we think that engineers will not accept literally the laboratory teachings that a dry concrete is less pervious than a plastic or fluid one.

In general, the laboratory does not take account of a great many factors which in practice are often preponderant. This discrepancy is particularly striking with regard to permeability tests. In attempting to produce water tight structures, it is better to guard against possible defects by increasing the amount of cement slightly than by reducing the quantity of mixing water.

BY R. W. CARLSON*

The authors have presented the most adequate treatise on permeability of concrete that has been published thus far. Their paper is in line with the growing attention to permeability of concrete for important structures. Therein may lie the solution of many mass-concrete problems. In particular, the aim should be to obtain an impermeable concrete which has a minimum potential volume change due to temperature, moisture, and other changes. In general, this means attempting to secure an impermeable concrete with the minimum cement and water content.

One of the outstanding facts demonstrated by the authors is that the hydration of cement in concrete may reduce the rate of water percolation by ten thousand times. To understand this clearly, one must appreciate the remarkable manner in which cement hydrates to

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form gel, principally calcium silicate gel. Some discussion of the gel may, therefore, be pertinent.

Studies at the Massachusetts Institute of Technology indicate that this gel is porous and sponge like, but not like gelatin gel, for it has a rigid structure such that when it is dried, it maintains its size and shape, except for a relatively small contraction, and the moisture merely leaves the pores. Thus, while the absolute solid volume of the gel is only a fraction larger than the solid volume of the crystalline silicates in the cement grain from which the gel was produced, the actual gross volume of the gel may be two or three times the original volume of the silicates from which it was produced. This is important because the pores in the gel are ordinarily so small as neither to permit appreciable percolation of water nor to permit contained water to freeze. In fact, if the gel could be magnified a thousand diameters, the pores would still be too small to visualize. The pore sizes can only be determined indirectly, from capillary behavior, for example, and are therefore subject to the uncertainty of assumptions which must be made. It is clear, however, that the pores exist and that they are too small to be visible even in the highest powered microscopes. The pore space in good concrete as determined by the ordinary absorption test is made up largely of the minute pores in the gel.

A remarkable property of the gel seems to be its ability to grow out from the cement grain into whatever inter-particle pores which may be present. Petrographic studies have indicated that a cement particle does not increase in diameter symmetrically upon hydration, but that the gel extends from the particle into what space is available without exerting any substantial expansion pressure.

The growth of the gel from a cement particle is limited and therefore only reasonably small pores can be filled by the gel. Petrographic studies indicate that while the growth of gel continues for months or years when moisture is present, in the usual 28-day period, the gel is able to fill completely inter particle pores of about 10 or 15 microns in diameter (less than one thousandth inch).

BY PAUL T. NORTON, JR.* AND D. H. PLETTA†

While appreciating the invitation to discuss this excellent paper, we have found it necessary to make our discussion general rather than specific because of the great difference between the purposes and

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[1]"The Permeability of Gravel Concrete," JOURNAL Amer. Concrete Inst., May, 1931; *Proceedings* Vol. 27, p. 1093.

methods of this project and of our own.[1] We are particularly interested in Fig. 2 and 3 and the statement that "inflow and outflow measurements after correction showed approximate agreement"; also the reasons why inflow measurements were used in this project in computing permeability coefficients. In the present project apparently there was always sufficient visible outflow to make it possible to use the outflow method, but the outflow method would fail completely in cases where there was no visible outflow because the outflow was less than the maximum evaporation, and this vapor would be lost each time the jar was removed from the funnel for successive measurements. The inflow method has one great advantage. With this method a number of mixes of low permeability, none of them having visible outflow, may be compared at any age and with any curing method. In our own tests we found great differences between the inflow of various mixes, none of which had any visible outflow. Investigators using the outflow method have sometimes found it necessary, therefore, to use either very lean mixes or make the tests at a very early age to secure an outflow that could be satisfactorily measured.

The analysis of the effect of length of specimen and end condition is very interesting and may explain some of the differences in the results obtained by various investigators. In our own project the ends of the specimens were wire brushed as they were removed from the molds 24 hours after casting. If the "U. of Wis. Gravel Test" data shown in Fig. 9 refer to our work, there are apparently some errors in this table, for none of our specimens were 12 in. long (unless this length has been corrected for a 3 in. length due to end condition at each end) and the specimens tested at 39 days were subjected to only 40 p.s.i. We should be glad to know whether these particular data in Fig. 9 refer to our work, and if so how the conversion factors were determined.

In our own work, with permeability specimens approximately 9½ in. in diameter, permeability results were much more erratic for mixes with 2 in. maximum aggregate than for mixes with 1½ in. maximum aggregate. We understand that in the present project specimens 18 in. in diameter were cast with 9 in. maximum aggregate and are wondering whether any special procedure was used in placing the concrete in the molds.

There is also doubt in our minds as to how the coefficients K_v and K_p were computed. In dividing these coefficients by the absolute volume of the mixing water and paste respectively, was the volume that for actual specimen or for a cubic foot of concrete?

The absorption curve in Fig. 2 increases for about 300 hours. We are wondering whether this test was made under pressure, for in our tests dried specimens attained constant weight after about 50 hours, when merely immersed.

The authors deserve to be complimented on this piece of research, especially for the way in which they have applied their results to a practical job.

(*To be concluded in November-December* JOURNAL)

Discussion of a paper by Messrs. Blanks and McNamara:

"MASS CONCRETE TESTS IN LARGE CYLINDERS"

Publication of this discussion, now closed, is postponed. Readers are referred to the JOURNAL *for November-December, 1935.*—EDITOR

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Competition for designs for working-class flats in reinforced concrete

Concrete and Constructional Engineering, Vol. XXX, No. 4, April 1935, pp. 218-231.

Reviewed by GLENN MURPHY.

Results of the competition for designs for working-class flats, promoted by the Cement Marketing Co. Ltd., and discussion of the winning design.

The statics of bridge design

G. DUNN, *Concrete and Constructional Engineering*, Vol. XXX, No. 5, May 1935, p. 285-292.

Reviewed by GLENN MURPHY.

Concluding number of a series, dealing with combined thrust and bending. Methods, supplemented by examples and charts, for designing circular or polygonal arch sections.

Charts for checking reinforced concrete sections

J. P. PORTER, *Concrete and Constructional Engineering*, Vol. XXX, No. 7, July 1935, p. 403-408.

Reviewed by GLENN MURPHY.

Charts showing the actual maximum concrete and tensile steel stresses plotted against M/bd^2 for varying ratios of compressive steel to tensile steel, for rectangular concrete beams in which $n = 15$.

A circular staircase

Concrete and Constructional Engineering, Vol. XXX, No. 6, June 1935, p. 344-345.

Reviewed by GLENN MURPHY.

Description of a reinforced concrete circular staircase with architectural details molded in situ. The structure is self supporting, and the steps and landings are cantilevered from an external hollow reinforced concrete wall. The internal diameter is 16 ft. and the height 34 ft.

The statics of bridge design

G. DUNN, *Concrete and Constructional Engineering*, Vol. XXX, No. 3, March 1935, pp. 175-191.

Reviewed by GLENN MURPHY.

Calculation of Rectangular Arch Sections, reinforced equally on the two faces. Methods, supplemented by examples and charts, for determining directly any one of the three variables (breadth, depth, and steel percentage) with the other two given.

Viaduct at Newport (Mon.)

Concrete and Constructional Engineering, Vol. XXX, No. 3, March 1935, pp. 169-174.
Reviewed by Glenn Murphy.

Design features of a 600 ft. by 50 ft. s-shaped viaduct on the main London-Fishguard road through Newport. The reinforcement consisted of I-beams, plates and bars, and was pre-erected, thus dispensing with shores and permitting normal use of the tracks below the viaduct during the construction period.

Wind tunnel at Farnborough

Concrete and Constructional Engineering, Vol. XXX, No. 5, May, 1935, p. 293-296.
Reviewed by Glenn Murphy.

Description of a wind tunnel constructed for the Air Ministry at the Royal Aircraft Establishment for making tests on aeroplanes up to 56 ft. span and 8000 lb. weight. The construction was complicated by the necessity for smooth transition lengths where the section changed from circular to square, and for a smooth jointless finish throughout.

The new code applied to design

C. E. Reynolds, *Concrete and Constructional Engineering*, Vol. XXX, No. 3, April 1935, pp. 232-243.
Reviewed by Glenn Murphy.

Continuation of a series. Discussion of general design of floor panels, calculation of bending moments, arrangement of reinforcement, beams, openings, partitions, and columns supporting flat slabs. Detail drawings of flat-slab construction, with calculation sheets for interior and end panels and internal and external columns are given.

Austrian fatigue tests of reinforced concrete beams

Engineering News-Record, Vol. 114, No. 20 May 16, 1935, p. 697. Reviewed by N. M. Newmark.

Brief statement of conclusions drawn by R. Saliger, Technical University of Vienna, from tests concerning influence on reinforced concrete beams of repeated load applications. Dr. Saliger's report has not yet been published. Apparently several million load changes, where the applied load was limited to 55 per cent of the breaking load, had very little effect on the ultimate strength of the beams.

Extensive rock grouting at Boulder Dam

Engineering News-Record, Vol. 114, No. 23, June 6, 1935, pp. 795-797. Reviewed by N. M. Newmark.

Particular attention was paid in the construction of Boulder Dam to the process of grouting under the dam and in the abutments. By means of holes drilled in the rock foundation, neat-cement grout was forced into fissures and seams at pressures up to 1,000 p.s.i. In the penstock and outlet tunnels grout was forced through radial holes drilled through the concrete lining in order to fill any voids behind the lining. A review of the details of the grouting procedure is given in this article.

The shear-area method

Horace B. Compton and Clayton O. Dohrenwend, *Proc. Am. Soc. C. E.*, May 1935, p. 605, Vol. 61, No. 5.
Reviewed by H. J. Gilkey

Explains and illustrates the use of the shear diagram for the solution of elastic functions of loaded beams in much the same manner as the moment diagram has been used in the method of moment areas. Certain advantages are pointed out. Such methods are especially useful in the solution of structural problems in concrete because of the indeterminate nature of many of the members and assemblies.

Practical notes on concrete making and placing

G. Dunn and A. W. Legat, *Concrete and Constructional Engineering*, Vol. XXX, No. 3, April 1935, pp. 244-262. Reviewed by Glenn Murphy.

Improvement in construction methods has perhaps not kept pace with improvement in design methods in recent years. Higher quality construction will promote use of concrete. Article summarizes information which will be an aid to better construction under the following heads: cement, sand, coarse aggregate, water, test cubes, proportioning, mixing, transportation, placing, curing and remedying defects.

Underground waters and concrete construction

F. W. Freise, *Concrete and Constructional Engineering*, Vol. XXX, No. 3, March 1935, pp. 163-168. Reviewed by Glenn Murphy.

Summary of field observations, supplemented by chemical analyses, over a 10-year period on the action of underground waters upon concrete. Apparently the observations were made in Brazil. The ingredients considered are: humic acid, lactic acid, carbonic acid, sulphuretted hydrogen, sulphates, and chlorides. A description of the action of each of these upon concrete, and recommendation of preventive measures is given.

Poles from centrifugal concrete

Paul Abeles, *Zeitschrift des Osterr. Ingenieur-und Architekten-Vereines*, No. 25-26, June 28, 1935, pp. 147-154. Reviewed by Inge Lyse.

The manufacture of centrifugally made concrete poles is described in detail, with sketches of reinforcement arrangements and types of form used. Detailed description is also given of tests made at the Vienna Technical Institute and of the results obtained. Careful observations were made of strains and crack openings. Durability tests were also conducted. All tests showed that the centrifugal method produces a concrete of high quality.

Cement macadam proves adaptable to unusual construction demands

Engineering News-Record, Vol. 114, No. 24, June 13, 1935, pp. 845-849. Reviewed by N. M. Newmark.

A cement macadam road at Camp Dix, N. J. built with salvaged sewage filter stone is described by Lt. William Whipple, Corps of Engineers, U. S. A., and an experimental cement-bound macadam pavement nearly a mile long near Camp Perry in Ottawa County, Ohio, is described by F. W. Stopher, Division Engineer, Ohio Dept. of Highways. Cores drilled from the Ohio pavement had an average compressive strength of about 2000 p.s.i. at 7 and 28 days.

Reinforced concrete bridge over the Seine

Concrete and Constructional Engineering, Vol. XXX, No. 5, May, 1935, p. 279-281. Reviewed by Glenn Murphy.

Design and construction features of bridge at La Roche-Guyon, said to be the largest arch bridge having a suspended floor. The main span is 528 ft. long with a rise of 75 ft. 6 in., and the width between parapets is 32 ft. 9 in. The ribs increase in depth from the springings to the crown to provide a maximum of flexibility, and to keep the maximum stresses approximately the same at all sections. The ribs are designed with a hollow cross section and spiral reinforcement. The design stress was 1280 p.s.i. for a concrete containing 670 lb. of cement per cu. yd.

Photo elastic determination of shrinkage stresses

Howard G. Smits, *Proc. Am. Soc. C. E.*, May 1935, p. 597, Vol. 61, No. 5. Reviewed by H. J. Gilkey.

Stresses were applied to the ground portion of a monolithic bakelite model of dam and ground. Lines of constant shear and the direction lines of principal stress were

determined photo-elastically. The set-up was idealized in that the dam and ground were assumed to be homogeneous and isotropic and the results are intended merely to give to the designer a general feeling for the distribution of shrinkage stresses in a concrete dam, rather than to supply a true picture of actual conditions. The tests show high stresses between dam and ground at the heel and toe of the dam.

French regulations for reinforced concrete

Les Annales des Pônts et Chaussees, III, March, 1935. Reviewed by B. MOREELL.

This issue of *Les Annales des Ponts et Chaussees* contains the new French Government regulations on reinforced concrete, promulgated by the Minister of Public Works on July 19, 1934. These regulations replace the regulations of October 20 1906. While there are a few differences, the regulations are substantially in accord with the code proposed by the Association of Constructors in Reinforced Concrete, which was discussed in detail by the reviewer in the A. C. I. JOURNAL for May-June, 1934. ("Observations on European Practice in Concrete Design and Construction," by B. Moreell, *Proceedings* Vol. 30, p. 391.

Progress of cement research during 1934

C. R. PLATZMANN, *Zement,* No. 29 and 30, July 18 and 25, 1935. Reviewed by INGE LYSE

A comprehensive review of all important studies of cement during 1934 is presnted. in this paper. The size of the field is realized from the fact that a total of 116 references in German, English and French languages are given. The field is subdivided into different subjects. Section I deals with standard specifications and their changes during 1934. Section II discusses cement testing and analyses of cements, while Section III presents a review of scientific researches. New cement mills are described in Section IV, and the properties of cement and effect of admixtures in Section V. The paper is of great interest and assistance to anyone interested in cement.

Design and tests of a vierendul truss of 28 meters span

R. L'HERMITE, *Le Constructeur de Ciment Arme,* Vol. 17, p. 97, May 1935. Reviewed by D. E. PARSONS.

The method of design and the testing of a reinforced concrete truss, approximately 92 ft. long, are described. The truss consists of 8 panels, with nine posts connecting the upper and lower chords. The posts are flared at the upper and lower ends sufficiently to include the diagonally placed tensile reinforcements which serve as the main ties. The positions of the points of inflection within each member surrounding a single panel are assumed in order to permit the estimation of secondary stresses. One-tenth scale models were tested, one being loaded at mid-span and another at a quarter point. The failures of the specimens are described.

Floor panel test of Isteg reinforcement

Concrete and Constructional Engineering, Vol. XXX, No. 6, June 1935, p. 350-352. Reviewed by GLENN MURPHY.

Comparative load-deflection tests of two floor panels during the construction of a building in London. Each panel was 13 ft. 5¼ in. by 13 ft. 6 in., and was constructed as a hollow tile floor with 3 in. ribs 15 in. o.c. The tiles were 12 in. by 12 in. by 5 in., and the thickness of the concrete slab above the tiles was 1½ in. In one panel each rib was reinforced with two ⅝-in. mild steel bars, while in the other panel the reinforcement consisted of two $\frac{11}{32}$ in. diameter Isteg bars. Deflections were measured under a load of 96.4 lb. per sq. ft. on each panel, but no large differences were noted in the behavior of the two panels.

Concrete placing methods on Chicago sewer tunnels

Engineering News-Record, Vol. 114, No. 26, June 27, 1935, pp. 911-914.
Reviewed by N. M. Newmark.

This is the last of three articles describing the construction of 27 miles of intercepting-sewer tunnels in Chicago. Previous articles appeared June 13 and June 20, 1935 in *Eng. News Record.* Tne tunnels vary from 17 x 17 ft. to 4 x 5 ft. inside of lining. This article describes the methods of placing the concrete lining developed by the various contractors. The form design consists solely of channel ribs behind which is built up longitudinal wood lagging just ahead of the rise of concrete. The form system is identical on all sections except for the method of supporting the ribs. The general construction plants and methods used by the different contractors are outlined.

Old-time continuous mixer used for concrete tunnel lining

Frank Rasmussen, Engineer, Link Belt Co., Chicago, Ill. *Engineering News-Record*, Vol. 115, No. 2, July 11, 1935, pp. 49-51.
Reviewed by N. M. Newmark.

In placing the concrete lining in the Chicago Avenue Water tunnel in Chicago, a belt conveyor distribution system is used in conjunction with an old-fashioned continuous paddle concrete mixer. Cement and crusher-run limestone aggregate are delivered to one end of the mixer in a steady stream, and mixed concrete is discharged continuously from the other end. Water is admitted to the mixer through a perforated pipe without measuring. The normal rate of discharge is 50 cu. yd. of concrete per hour, but frequently the rate is speeded up to 84 cu. yd. per hr. It is claimed that a satisfactory quality of concrete is produced, daily test cylinders having a strength of 2000 to 6700 p.s.i., a rather large variation. Details of the placing of the concrete are outlined.

Concrete product journal

Beton Stein Zeitung, Berlin, Vol. 1, Heft 1 and 2, July 1935. Reviewed by Inge Lyse.

This new magazine is devoted to the concrete products field and the two numbers of it which have appeared to date give a very promising indication of the type of articles which will be published. In No. 1 is included articles on "Grinding and Polishing Concrete Blocks and Terrazzo," "Considerations in Expansion of Concrete Products Plants," "Hollow Masonry Using Precast Materials," "Steam Curing of Concrete Blocks," and a number of brief articles and notes.

No. 2 contains "1850 years old concrete and its use as concrete product." "Coloring of cement roofing stone," "Economical work division in concrete products plants," articles on governmental regulations for the concrete products industry and smaller notes and statements.

The new Austrian standards for reinforced concrete

Staff Article, *Zeitschrift des Osterr. Ingenieur-und Architekten-Vereines*, No. 29-30, 1935, pp. 175-176.
Reviewed by Inge Lyse.

The new Austrian regulations for reinforced concrete contain several important changes. More rigid requirements are laid down for the manufacture of the concrete. The mixing water must be within one per cent of the specified amount. Attention must be given to gradation of aggregates and mortar content in the concrete and the consistency of the concrete must be observed in order to insure uniformity.

In the design of reinforced columns the so-called "addition" law is followed. However, the amount of strength contributed by the spiral is limited by a requirement of maximum two per cent spiral reinforcement. The efficiency of the spiral

reinforcement has been set at 2.0. Special high strength reinforcing steel is permitted with corresponding higher working stresses, that is, 50 per cent of the yield point of the steel with a maximum of 1800 kg. per sq. cm. (about 26,000 p.s.i.).

Reinforced concrete reservoir at Burnham

Concrete and Constructional Engineering, Vol. XXX, No. 3, March 1935, pp. 157-161.
Reviewed by GLENN MURPHY

Design and construction features of a 1,600,000 gal. reinforced concrete reservoir to serve as a balancing tank between pumps and mains, and as emergency supply for the Burnham, Dorney and Hitcham Waterworks Co. The reservoir is 180 by 120 by 12 ft. inside, with a central partition wall to allow cleaning and inspection of half the reservoirs at one time. The walls are of propped cantilever design, varying from 12 in. to 9 in. thick. The roof is a 6 in. mushroom-type slab, supported on 12 in. square columns 15 ft. o.c. The maximum design stresses for the structure were 16,000 and 600 p.s.i. for steel and concrete respectively. The floor was placed in two layers: the first, 3 in. of plain 1:3:6 concrete; the second, 6 in. of 1:1½:3 concrete reinforced with ⅜ in. bars 12 in. o.c. in both directions. No waterproofing material was used in the structure.

Restraint by torsion

ROLF SCHJODT, *Bulletin No. 8, 1934, Det KGL, Norske Videnskabers Setskabs Skrifter*, Trondhjem, 1935.
Reviewed by INGE LYSE.

This bulletin, published in German, presents an interesting theoretical study of the restraining moments introduced in reinforced concrete beams and slabs from their connections with girders and beams. Much emphasis is placed on the apparent reduction in the positive moments by these restraints and formulas are developed for the evaluation of the amount of rigidity produced by the torsional effect. This highly mathematical treatment is to a certain extent based on assumptions which to many engineers may be considered of questionable validity, and since no experimental evidence is offered, the results should not be accepted without due study. However, the bulletin is of considerable interest to those concerned with the strictly mathematical side of the torsion question, and its major importance probably lies in the fact that it will stimulate new interest in the experimental studies of the continuity and rigidity present in all reinforced concrete structures. The bulletin contains 69 pages with 12 figures.

Concrete repairs on an ocean pier

FRANK G. LEE and PAUL JORGENSON, Chief Engineer and Structural Engineer, St. Petersburg, Fla.
Engineering News-Record, Vol. 115, No. 1, July 4, 1935, pp.1-6. Reviewed by N. M. NEWMARK.

The Municipal Recreation Pier at St. Petersburg, Florida, an 1820 ft. reinforced concrete structure 100 ft. wide, built in 1925, had suffered serious deterioration by the summer of 1932. The structure had been erected without expansion joints. Expansion at construction joints, seepage of water ponded in a depressed section carrying a street car track, and scaling due to insufficient covering of reinforcing steel had badly damaged the concrete. The reconstruction and repairs necessary to remedy the damage are outlined in the article. Four expansion joints were installed and subsequent measurements of movement show the necessity for their installation. The area in the street-car track pocket was waterproofed and drained, all construction joints and other cracks were filled, and the defective members of the structure were repaired with gunite. Details of the repair work are described. The cost of the repairs is given as about $69,000 and the original cost of the structure about $1,000,000.

Determination of moisture in sand by means of "areometer" test

PER HALLSTROM, *Betong*, No. 2, p. 66, 1935. Reviewed by INGE LYSE.

This article describes the "areometer" test for determining the moisture in sand. This test is very simple and therefore suitable for field work. It depends upon the difference in the specific gravity of a liquid before and after sand has been introduced.

The necessary equipment consists of one 500 cc and one 50 cc graduate and a hydrometer. Red alcohol is the liquid used. The test procedure is as follows: 200 cc of red alcohol is placed in the 500 cc graduate and the specific gravity determined by the hydrometer. The sand is poured into the alcohol until the sand reaches the 300 cc mark. Enough excess alcohol for a hydrometer determination is poured from the 500 cc graduate into the 50 cc graduate and the difference in specific gravity between the pure alcohol and the alcohol containing the moisture carried by the sand gives amount of water in the sand. It is recommended that the water content be obtained from experimental curves instead of using calculated theoretical values which contain certain errors. The test should be carried out at a fairly constant temperature.

Shrinkage cracks in reinforced concrete tanks

G. P. MANNING, *Concrete and Constructional Engineering*, Vol. XXX, No. 5, May 1935, p. 271-278. Reviewed by GLENN MURPHY.

Discussion based on observations of recently constructed tanks and theoretical considerations of linear shrinkage. Tank floors seem immune from visible shrinkage cracks, tank walls show vertical cracks, and tank roofs show cracks across the corners. Shrinkage effects may be divided into temperature-shrinkage resulting from chemical reactions involved in setting, and time-shrinkage. The resulting tension may be superimposed upon structural tension, temperature stresses, and possible effects due to differences in moisture content. Most cracks result from temperature fall within 48 hours after placing. Preventative recommendations are: Use of a low heat cement without sacrifice of early strength, use of thinner sections to reduce temperature rise, use of insulating materials to prevent temperature fall until the concrete develops some tensile strength, use of heat-radiating shutter to reduce temperature rise, use of a thick foundation slab or building tanks on flexible columns, make the walls and roof independent of one another, use curved walls, use a cement having less shrinkage, diminish the time interval between lifts of concreting, use smaller tanks.

High speed in concreting at Norris Dam

Engineering News-Record, Vol. 114, No. 20, May 16, 1935, pp. 698-701. Reviewed by N. M. NEWMARK.

During the months of January, February, and March, the average rate of placing of concrete in Norris Dam was 3,498 cu. yd. per 23-hr. day, with a maximum of 4160 cu. yd. in one day. The maximum rate, equivalent to 180 cu. yd. per hour, was attained with three 3-yd. mixers and two 1900-ft. span cableways, each handling a 6-yd. bucket. Although it was expected that only one cableway could be used in the narrow sections toward the top of the dam, it has been found that very little time is lost with both cableways operating, since the shifting of the head-towers sideways during travel of the buckets between loading and discharge points permits both cableways to deliver concrete to the same spot. It is estimated that three months will be saved in the concrete placement.

The form construction and handling are described in the article. An unusual feature is the elimination of the vertical steps for keying successive lifts of concrete.

The surface of each lift is laid on a 5 per cent grade descending from downstream to upstream faces. Smooth finishing of the graded surface permits the use of wheeled portable equipment on the hardened concrete of each lift.

Concreting in cold and freezing weather

GROEBL, *Zement*, Vol. 24, No. 24, June 13, 1935, p. 368-370. Reviewed by INGE LYSE.

Recommendations for concreting at temperatures of 41° F. to 32° F. and at freezing temperatures are presented as a guide for construction work. The concrete should be so protected that it will be able to resist the destructive forces caused by freezing. It is recommended that the concrete in ordinary structures should attain a strength of 1400 p.s.i., and in hydraulic structures 2100 p.s.i. before being exposed to freezing temperatures. For concreting in cold weather high-early-strength cements are usually found superior to regular cement. Warm water is recommended for mixing water while heating of aggregates is generally unnecessary. Detrimental results from freezing will generally be avoided if the concrete is protected against temperatures of less than 36° F. for the first 72 hours. The aggregates should be protected against freezing and provision made for heating aggregates where low temperatures prevail. While certain admixtures are permitted for cold weather concreting they should not be depended upon for important work. The reinforcement and forms must be free from snow and ice at the time of placing the concrete. The use of cement with high heat of hydration does not in any way lower the requirements for proper precautions when concreting in cold weather.

Cost of raising settled road slabs

CONCRETE, Vol. 43, No. 5, May, 1935, p. 10—from HIGHWAY RESEARCH ABSTRACTS.

The cost data were furnished by the Illinois Division of Highways and cover the operation of ten mud-pump outfits during 1934 on State highways. The costs include labor, field supervision, gasoline, oil, repairs, moving, field expense, and field storage. No machinery rental or depreciation are included. Equipment for each outfit consisted of one mud pump, one 200 or 300 gallon water tank on a trailer, one or more water supply tanks for use at the site, two trucks to haul earth, one truck for general utility and water hauling, one air compressor of 120 to 160 cubic feet capacity, one paving breaker, and one or two drills. The crew usually employed eight or nine men.

Number of settlements raised	2,484
Square yards of pavement raised	202,046
Average direct cost per square yard	$ 0.296
Cubic yards of depressions filled	6,835
Cost per cubic yard	$ 8.78
Number of holes bored in pavement	50,753
Average cost per hole	$ 0.11
Pavement expansion joints cut	.605
Cost per joint (Average length 18 ft.)	$ 3.59
Cubic yard of earth pumped	12,578
Cost per cubic yard, including hauling	$ 2.88
Cement used, sacks	8,053
Cost per sack	$ 0.46
Square yards asphalt patches removed at settlements	25,673
Cost per square yard (removing asphalt)	$ 0.11
Total direct cost for 1934	$59,996.48

New combination structural steel and reinforced concrete floor design called Gamma

LeGenie Civil, Vol. CVII, No. 4, July 27, 1935, p. 93. Reviewed by R. L. BERTIN.

The construction consists of: (1) Reinforced concrete columns; (2) Structural steel beams to the top of which steel spirals are welded which rest on the reinforced concrete columns in two directions, thus forming a series of rectangles; (3) Two-way

reinforced concrete slabs bearing on the top of the structural steel beams and encasing the spirals.

The advantages claimed for the system by the inventor, M. L. Gellusseau are: (1) Simple structural steel work through the elimination of connections from beams to columns; (2) Greatly reduced cost of form work; (3) Elimination of intermediate beams through the use of large two-way slabs which may be of uniform thickness, or slightly crowned in the form of vaults; (4) concealment of steel beams within walls and partitions; (5) Sound insulation through the use of cork flooring or cork concrete on top of the slab; (6) Fire resistance equal to that of reinforced concrete structures when the "Alpha" beams are encased in masonry.

The continuity, rigidity of the joints, perfect joining of the steel beams to the concrete slab, as well as the resistance to flexure of the assembly were demonstrated by tests to conform to the laws of elasticity.

The patented structural steel beams with the welded spiral are designated as Alpha beams.

The quality factor and the controlled concrete construction

W. Neuffer, *Zement*, No. 27 and 28, July 4 and July 11, 1935. Reviewed by Inge Lyse.

A discussion of the relative merits of concrete as compared to steel and wood. The quality factor is expressed as the ratio between strength of material and its unit weight or as the ratio between permitted working stress and the unit weight. Comparisons are also made between the factors of safety of different materials and variations in the factor of safety of concrete in different countries. It is noted that while Germany requires a factor of safety of as much as 5.62, England requires 4.08, France 2.80 and Switzerland as low as 2.67. Concrete strength as high as 6400 p.s.i. has successfully been obtained in the field with results from fusion cement as high as 7000 p.s.i. at 28 days and 8500 p.s.i. at 90 days.

The quality of the concrete is determined principally by the cement. Aggregate, method of manufacture and water content are also important. The following formula is recommended for estimating the strength of concrete at 28 days:

$$fc'_{28} — 1.0\ C + 1000\ (R\text{-}2.14) — 1.5W$$

where C is cement content in kg per cu. m.

R is unit weight of concrete at 28 days (ranging between 2.24 and 2.37)

W is water content in liters per cu. yd.

Recommendations are given for how to obtain the best results in concrete control and constructions, and pleas made for a reduction in the factor of safety required by German regulations.

The use of vibrated concrete for repairing concrete structures

Ch. Tournay, *LeGenie Civil*, Vol. CVII, No. 2, July 13, 1935, pp. 39 to 43. Reviewed by R. L. Bertin.

The author brings out the advantages of vibrated concrete over the usual methods of repairs, namely cement mortar application by trowel or gunite for repairs of concrete structures which have become damaged through exposure to the elements or chemicals, shocks, etc.

When it is desired to bond new concrete to old, it is essential that the shrinkage of the new concrete be reduced to a minimum by reducing the proportions of cement, fines and water. It is precisely these three things that vibration reduces materially.

Damage generally occurs where the dense and complicated reinforcement interfered originally with proper placing of the concrete. In repairing such sections, the application of vibration insures a uniformly dense and compact emplacement.

Vibration of concrete is suitable not only to the repair of structures but to strengthen existing structures, either through the encasement of new reinforcing bars or the replacement of a weak concrete by one of greater strength where compression stresses occur.

The author describes in great detail repairs of various reinforced concrete structures by means of vibrated concrete, as well as the encasement of structural steel frames subjected to more or less intense oxidization.

The results of a study of the properties of vibrated concrete as affected by grading of the aggregates, the characteristics of the gravel, temperature, fineness of the cement and the water cement ratio are given in the form of tables and graphs.

The great "dock-entrance lock" of Saint Nazaire and the supplementary structures

MELCHOIR DE LISLE, Engineer cf Ponts et Chaussees. *Les Annales des Ponts et Chaussees* I, January, 1935. Reviewed by B. MOREELL.

This article describes the details of the combined lock and dry dock which was constructed at St. Nazaire to provide for the fitting-out of the great liner Normandie. The lock is approximately 1148 ft. long, 164 ft. wide at the entrance and has a depth over the sill at mean high tide of approximately 40 ft. There is a sliding gate at each end, capable of being operated at any stage of the tide, and with the higher water level on either side of the gate.

The body of the dock consists of mass concrete walls resting on the natural rock, the floor of the dock being a 12 in. thickness of concrete paving on natural rock. The side walls were constructed of slag-cement concrete up to mean tide level and above this level are of high alumina-cement concrete. The five lines of keel-blocks are of reinforced concrete. The entrance sections are of heavily reinforced concrete with granite seats for the gates, which have gaskets of "greenheart."

The quay walls which connect the new lock with the existing quay wall structures are the relieving platform type constructed of mass portland-cement concrete. At the front of each quay wall there is a line of heavy, reinforced concrete sheet piles, made of high alumina cement concrete.

The interesting features of this work are, (a) the use of blast furnace slag cement for mass concrete which will be kept moist, (b) the preference for mass concrete for the heavy dock walls, (c) the use of high alumina cement for heavy reinforced piles subjected to sea-water action.

Notes on the construction of a reinforced-concrete bridge over the River Lot, at Luzech

M. CAZES, Engineer of Ponts et Chaussees, *Les Annales des Ponts et Chaussees*, II, February, 1935. Reviewed by B. MOREELL.

Describes the construction of an arch highway bridge having three spans of approximately 125 ft. each, replacing an old steel arch bridge not capable of carrying modern traffic. The two river piers of the old bridge were utilized, without strengthening, to support the new arches. The old abutments were also utilized but were strengthened. To obtain suitable connections with the existing road grades, it was necessary to keep the rise of the arches as small as possible. On the other hand, the necessity for increasing the free water-way required a high rise and small length of arch rib. To satisfy both of these conditions as nearly as practicable, the engineers decided to use an arch with rise of approximately ⅛ of the span and to utilize high alumina cement with very high working stresses for the arch rib. The working stress in

compression for standard portland cement concrete was 1000 p.s.i., while for high alumina cement concrete it was 1700 p.s.i.

To avoid imposing lateral thrusts on the river piers under a condition of non-symmetrical loading, it was decided to design the three arch spans as continuous over the river piers. Cast steel expansion rollers supported the arches on the piers. At the abutments, the arches are supported on Mesnager articulations.

Before opening the bridge to traffic it was tested under working loads consisting of trucks loaded with sand. Measurements of deflections and rotations were made. The measured deflections did not check the calculated deflections, the latter being much larger. The author concludes that methods used for calculating stresses and strains in homogeneous materials are not strictly applicable to reinforced concrete.

Cost data and the fundamental formulas used in calculating the stresses in the arches are given.

Grouting contraction joints in Hogan dam

S. A. Thomas, Jr., *Civil Engineering*, Vol. 5, No. 5, May, 1935, p. 296 Reviewed by J. R. Shank.

The Hogan Dam is a concrete arch dam on the Calaveras River in Calaveras County, Calif., and was built for flood control. The dimensions of the arch for length, height, width at crest, and width at base are 620, 115, 15, and 50 ft., respectively. The spacing of joints is about 40 ft. "As grout ducts for the joints, 2-in. half-round metal grout pipe was used, one half being placed on the forms and the other half being sealed to the first half when the forms were removed." The grout piping for each joint consisted of three horizontal feeders at different levels having vertical laterals above and below.

Grouting was carried out one and one-half years after construction. The procedure was to fill a joint completely with water under some pressure, calk up all leaks with lead wool, and check up the piping at each inlet opening. The water was then let out and blown out with air and grouting was started at the bottom piping, progressing upward until good grout appeared at the crest. Pressure was then applied and held until the grout stiffened. The grout mix ranged from 15 to 8 gal. of water per cu. ft. of finely ground cement. The thinner grout was used at starting and for very thin openings. The mix was thickened to the 8-gal. mix as soon as possible. Pressures ranged from 10 to 90 p.s.i. The lower pressures were used for the first series of joints. It was found possible to move some of the sections upstream by increasing the grout pressure. The amount of grout placed usually exceeded the amount calculated for the joint. The aggregate top joint opening amounted to 2 in., which was ¾ in. more than when grouting was started. The average shrinkage at the crest amounted to 0.00025 in. per in.

Some suggestions resulting from this experience are:

1. Grout stops should be designed for high pressures, as much as 300 p.s.i., so that the sections and joints may be adjusted at grouting.
2. High dams might be grouted in sections vertically, using horizontal grout stops.
3. Grouts using more than 8 gal. per sack of cement should be avoided.

Manual of instructions for concrete inspectors

Published by Metropolitan Water District of Southern California, Los Angeles, 59 pp., Pocket size 75 cents. Reviewed by J. W. Kelly.

This manual outlines briefly the approved concrete inspection practice of the Metropolitan Water District of Southern California relating to the construction of the Colorado River Aqueduct. It is supplementary to the specifications and is intended to be used as a book of reference on matters not covered in detail by the specifications.

The manual was made necessary by the fact that the construction is proceeding at widely separated points, extending over a distance of 240 miles. Its use is expected to result in uniform inspection practice throughout the length of the aqueduct.

A feature of the manual is its definition of the responsibility of the various employees, particularly with regard to the testing engineer and the various division engineers. Definite instructions are given to the inspector regarding his responsibilities and his relations with the contractor; and undue interference with the contractor's methods is forbidden.

The work of concrete inspection is divided into (a) plant inspection, and (b) placing inspection. The plant inspector inspects all materials, makes field tests of aggregate, and controls the proportioning, batching, mixing, and conveying of concrete. The placing inspector controls the time of concreting (concreting is forbidden whenever the air temperature exceeds a specified limit); he inspects the preparation of the subgrade, the setting of forms and reinforcement, the installation of joints and fixtures, the conveying of concrete at the site of the work, the compacting, finishing, and curing of concrete and the repair and protection of finished work; he controls the consistency of concrete; and he makes concrete specimens for laboratory test. Various general duties of inspection such as those covering excavation, plant and equipment, weather observations, and cleaning up are to be divided between the plant and placing inspectors by the division engineer, according to local conditions. Standard report forms minimize the amount of paperwork in the field, and serve as a convenient check for the inspector as to the completeness of his coverage of the day's duties.

The manual covers the following subjects: Administrative and general, inspection of materials (except aggregates), inspection and testing of concrete aggregates, inspection and testing of concrete, inspection of placing of concrete, and weather observations.

Analysis of continuous arch systems and theorem of nine displacements

KOZABURO MISE, Professor of Civil Engineering, Kyushu Imperial University, Japan. Reprinted from the *Memoirs* of the Faculty of Engineering, Kyushu Imperial University, Vol. VII, No. 5, 1935.

Reviewed by CHARLES S. WHITNEY

In a 68-page booklet with 16 plates, the author presents in English, two methods of analysing continuous arch systems together with a number of numerical examples which indicate the effect pier elasticity and the relative accuracy of the two methods for systems of different proportions.

The author's "Theorem of Nine Displacements" which he terms "an exact and complete solution of continuous arches on elastic piers" leads to formulas which express the general relation between the nine displacement components at the three consecutive supports and the loads on the two included spans and pier under consideration. By treating successive pairs of spans with their intermediate pier a solution may be found for any system of arches regardless of condition of loading, size or arrangement of piers, or character of foundations. The solution is considered exact because the formulas include the effects of displacements due to moments, direct thrusts and shearing stresses produced by loads and temperature changes.

By eliminating the effect of strains due to direct thrust and shear, and considering the two displacements at each of three consecutive pier heads, the Theorem of Nine Displacements is reduced to the "Theorem of Six Displacements," an approximate

method applicable to ordinary arch systems. The latter method is fundamentally the same as Ostenfeld's deformation method ("Die Deformationsmethode" by A. Ostenfeld, Julius Springer, 1926) and the slope deflection method of the Committee on Concrete and Reinforced Concrete Arches of the American Society of Civil Engineers. ("Final Report of the Committee on Concrete and Reinforced Concrete Arches," Transactions Am. Soc. Civil Engineers, 1935.)

A very valuable portion of the paper is the solution of various two- three- and five-span systems of different proportions with complete influence lines.

One of the author's conclusions is that "as a result of the examination of those influence lines, attention should be called particularly to the fact that the effect of the pier elasticity on the stresses in the arch ribs is quite remarkable, even in an arch system with short piers; and the usual solution of the fixed arch which gives values too remote from the actual stresses could not, even as a practical approximation, be used with safety in the design of continuous arch systems." A study of the influence lines will help the designer form his own opinion as to importance of pier elasticity in any particular case.

To test the limitations of the approximate method of analysis, the author solved by the two methods nine different three-span arch systems with varying pier heights and arch rise ratios. He shows that the error of the approximate method is not materially affected by the pier height but that the error increases as the arches become flatter. He concludes that the approximate method (theorem of six displacements) is universally applicable to ordinary continuous spans with elastic piers and will give reasonable and sufficient results, provided the rise ratio is not too small (say not less than $\frac{1}{8}$). It appears to the writer that the approximate method will give sufficiently accurate results for any practical continuous arch system provided that for very flat arches the rib-shortening effect be calculated for live loads, shrinkage, and temperature changes as well as for dead loads.

Professor Misé's work represents a tremendous amount of labor and is an important contribution to the subject.

The International exposition in Brussels

P. Calfas, *LeGenie Civil*, Vol. CVI, No. 18, May 4, 1935, pp. 425-431. Reviewed by R. L. Bertin.

Gives a detailed description of the layout of the exposition and the 140 palaces and pavilions comprising it.

Several of the more important buildings are permanent structures. The largest and most remarkable, from the standpoint of construction is the Grand Palace. This building is devoted to the exhibition of transportation presented in the form of a model railroad station. The building is 160 m. by 90 m. without any interior columns.

The framing consists of 12 three-hinged reinforced concrete arches, spanning 86 m. with a rise of 30 m.

The section of the arches is rectangular 1 m. thick throughout; the height being 1.5 m. at the spring line, 1.80 m. at the haunches and 1 m. at the crown. The hinges are of chrome nickel steel and cast iron.

The axis of the arch is three centered. This shape was selected because it minimizes the bending moments resulting from the roof loads, concentrated at intervals of 4.80 m., and the wind loads.

The front and rear of the building are self carrying walls. Between them are six independent sections, 24 m. long, consisting of two arches spaced 12 m. on centers,

braced by cross beams cantilevered 6 m. on either side of the arches. The cross beams are spaced approximately 4.80 m. across the arch span and carry vertical windows 3.95 m. high, and a slightly sloped horizontal concrete slab forming the roof from the top of the windows to the beam. This construction gives the structure the appearance of a gigantic stair, the steps of which are outlined by the vertical windows and the horizontal roof slabs.

The arches bear on concrete foundations 7 m. x 6.52 m. in section and from 1.1 m. to 2.4 m. thick, carried on 29 piles, four of which are vertical and 25 inclined at an angle of 25 deg. The thrust of each arch is about 850 tons.

The reinforcement for the arches was assembled on the ground in six sections, weighing 10 tons each; hoisted to the top of the arch by a specially constructed crane and slid down the forms to their position; the ends of the bars of the adjacent sections were butt welded, eliminating all hooking and lapping of the main reinforcing bars.

The form work consisted of a complete unit for two arches and the cross beams, supported on four 3-hinged steel arches the extrados of which matched the intrados of the concrete arch. The contact forms were made of sheet metal stiffened with U-irons properly braced. The total weight of the unit was 600 tons.

The concrete was conveyed in place by means of two concrete pumps which forced the concrete through a 150 mm. diameter pipe, a distance of 200 m. and a height of 40 m., at the rate of 13 m^3. per hour per pump. The total concrete per group was 750 cu. m. and required 29 hours or practically two days to put in place.

The concrete was compacted by means of electric vibrators acting on the forms, the reinforcement and in the mass itself. High early strength cement was used and permitted the forms to be stripped in 12 days. A pressure of 300 tons was applied by means of jacks at the crown which spread the ends of the arch sections at the crown 120 mm. and raised them 80 mm. These measurements checked very closely with the conputed values. The steel arches were then lowered enough to clear the concrete arches and rolled to the next position and reset for the next section. The first section was stripped May 1, 1934 and the sixth and last one at the beginning of October. (A view of this structure may be seen in *Architectural Concrete*, Vol. 1, No. 3, published by the Portland Cement Association.)

Cost of maintenance of highways of the New Jersey State Highway Department

Sigvold Johannesson, Designing Engineer. From an Unpublished Report of the New Jersey State Highway Department. Reviewed by Highway Research Board

Since 1921 the New Jersey State Highway Department has kept complete records of the cost of maintenance of the highways within its jurisdiction. The records divide the maintenance into eight items as follows:

A. Ordinary Maintenance—1. Work on surface-traveled width; 2. Work on paved shoulders; 3. Work on unpaved shoulders, drains and ditches; 4. Work on guard rails and fences.

B. Extraordinary Repairs—5. Resurfacing, rebuilding, heavy patching of main pavement; 6. Rebuilding paved shoulders; 7. Rebuilding unpaved shoulders and drains, new ditching; 8. Replacing guard rails and fences.

The maintenance cost of each of these items includes the cost of all labor and material, a proportional part of the cost of all equipment and tools and of salaries and office expenses of the Maintenance Department. They do not include the cost of

cleaning, snow removal, lighting, bridge maintenance and operation, and other similar items which are kept separately and are not included in this discussion.

The greater part of the New Jersey State Highways are paved with reinforced concrete and there are sufficient records to present maintenance costs of such pavements. Other types of pavement in the State are considerably more limited, and the records for them show the trend rather than the true average costs of maintenance.

TABLE 1—AVERAGE ANNUAL COST OF MAINTENANCE OF PAVEMENTS PER 100 SQUARE YARDS

Type of Pavement	Length Miles	Area Square Yards	Vehicles Per Day Average	Cost
Reinforced concrete	736.1	10,548,300	4,700	$ 0.70
Sheet asphalt, concrete base	25.8	459,000	4,600	1.34
Bituminous concrete, concrete base	75.1	1,273,800	7,000	2.13
Plain concrete (prior to 1923)	122.8	1,438,600	2,800	2.55
Sheet asphalt, macadam base	37.9	459,600	2,900	3.86
Bituminous penetration macadam	12.6	218,700	2,700	5.53
Gravel	57.1	536,100	1,000	6.49
Bituminous concrete, macadam base	67.9	894,300	6,400	8.34
Macadam	14.8	170,600	3,000	13.28

TABLE 2—REINFORCED CONCRETE PAVEMENT—AVERAGE ANNUAL COST OF MAINTENANCE PER 100 SQUARE YARDS

Maintenance Started. Year	Length Miles	Area Square Yards	Vehicles Per Day Average	Cost
1923	30.7	383,780	2,400	$1.29
1924	76.9	1,124,580	3,800	0.56
1925	93.6	1,231,290	3,400	0.57
1926	67.3	870,330	3,500	0.81
1927	15.9	244,260	7,800	0.48
1928	121.4	1,620,830	3,600	0.43
1929	104.7	1,572,960	4,500	0.84
1930	118.1	1,827.110	7,000	0.86
1931	107.5	1,673.130	5,800	0.70

Current Research

Thin plates for web reinforcing

A summary of unpublished information from J. T. THOMPSON, Professor of Civil Engineering, Johns Hopkins University.

During 1934-1935 an investigation of the practicability of using thin steel web plates in reinforced concrete beams instead of conventional stirrup arrangements was made in the Civil Engineering Department of The Johns Hopkins University. Three series of heavily reinforced beams, designed to fail in shear, were constructed and tested. The first series consisted of beams without web reinforcement, the second of beams reinforced with vertical "U" stirrups of orthodox design, and the third of beams having web reinforcement of steel plates ranging in thickness from $\frac{1}{64}$ to $\frac{1}{16}$ in., some perforated and some unperforated. These plates were 11 in. deep and extended throughout the outer thirds of the beams. (See sketch.) The beams were 8 in x 14 in. x 5 ft. center to center of bearings. The concrete strength and uniformity were controlled satisfactorily by the water-cement ratio method. The beams were loaded at the third points.

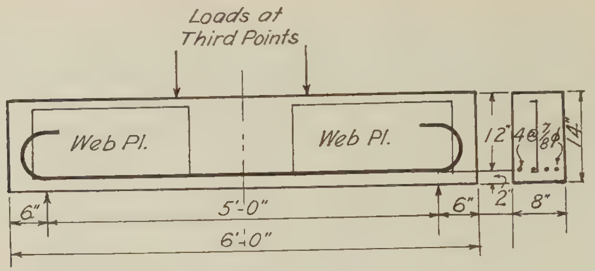

FIG. 1

The load-deflection curves for all beams were much alike. If anything, the beams with web plates were a little stiffer, that is to say deflected a trifle less for a given load, than did those of the other two series.

The load at the first occurrence of diagonal tension cracks was practically the same for all series. Beyond this point, however, beams with web reinforcement were superior to those without such reinforcement, particularly as regards their capacity to endure loads beyond the point where the deflections began to increase noticeably for constant increments of applied load. In contrasting the stirrup beams with the web plate beams it can be said that while there is but little difference in the ultimate strengths observed, the weight of steel in the webs of the latter may safely be reduced to about half of that in the former. In addition, it is believed that from the viewpoint of construction simplicity and ease the use of web plates offers a distinct advantage.

Huggenberger strain gauges attached to stirrups indicated negligible stress until the first appearance of diagonal tension cracks. Those attached to web plates indicated strain roughly proportional to the applied load when the gauge was inclined at the angle computed for maximum values of principal tensile stress.

By attaching Huggenberger gauges to the web plates and providing for their rotation it was possible to measure the principal stresses at various angles to the horizontal. When the beam section was reduced to a homogeneous one by conversion of the steel to comparable concrete areas, a very pretty check was obtained between theoretical and observed principal stresses. Of course it was necessary to determine the modulus of elasticity of the concrete. This was done when the control cylinders were tested for strength.

A marked tendency for beams reinforced with plates to split or separate into two halves at the plane of the plates was observed at ultimate loads. This was accompanied by a buckling of the upper edges of the plates. It was attributed to the influence of the hooks at the ends of the main reinforcement which applied a longitudinal load to the block of concrete between the diagonal tension crack and the beam end. This load was directed toward the beam center-line and appeared to be unequally applied on the two sides of the web plate, the differential in such load being apparently responsible for the splitting mentioned.

It is planned to extend the study further, particularly with a view of eliminating this separation. It is believed that doing away with the hooks will prevent splitting.

The Mechanics of Plastic Flow of Concrete*

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Research work in the field of plastic flow of concrete, up to this time has been confined largely to getting data on the phenomenon in general and on the effects of different materials, mixtures, and surrounding conditions. The data have usually been presented in curves, plotted with plastic flow deformations as ordinates and time as abscissas. Groupings of these curves have been plotted and conclusions drawn, which were little more quantitative in nature than visual inspections of the curve sheets would give. In only a few cases have curve equations been presented for these graphic expressions. It is the purpose of this paper to analyze many of the data available and present them as curve equations, and to present methods and formulas for calculating deformations and unit stresses in reinforced concrete members.

The most noted examples of curve equations are those by L. G. Straub (16)[1] and F. G. Thomas (20). The Straub equation is as follows:

$\rho = K\sigma^p t^q$ where

ρ = the unit plastic deformation after a time

K = a coefficient to be obtained from tests

t = the time

σ = the unit stress

p and q are exponents, which depend on the elastic conditions at the time of loading.

A valuable development by Thomas is the discovery that the deformation time curve for plastic flow approaches a limiting ordinate or asymptote line. He developed a curve which expresses "Ratio of the limiting value of creep to the creep during the first year under load" as ordinates and age at loading in months as abscissas. The following equation fits very closely the curve as he drew it.

$$y = 1 + 0.315\sqrt[3]{x}$$

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[1]See references in appended bibliography.

His last calculated point is at three months. It will be noticed that the limiting value increases with the age at loading.

The work of Thomas is a valuable contribution but the expression by Straub is more nearly in line with what can be developed into something which may be of use to the engineer. The term σ with its exponent p indicates that it may be that plastic flow is *not* directly proportional to the unit stress imposed and test data can be found that bear this out; however. tests at the University of California (11, 12, 13 and 14) on what may be said to be ordinary portland cement concrete, as well as tests at the Ohio State University and at The Bureau of Building Research, England (15 and 20), produce data, which indicate that for ordinary conditions the plastic flow *is* proportional to the stress. If this proportionality principle is accepted, the formula by Straub might be reduced to this

$y = Kt^q$ where y now represents the plastic flow for a unit stress of unity.

REDUCTION OF DATA TO COVER EQUATIONS

The most prolific sources of data on plastic flow of concrete are the University of California (11, 12, 13, and 14); the Bureau of Building Research of Great Britain, (15 and 20); and The Ohio State University. More than one hundred test curves from these three sources were studied and equations like this modified Straub equation were fitted to them, and the terms K and q, or C and a respectively as used in this paper, were found.

Table 1 shows these data together with data concerning sources, materials used, mixes, elastic properties, unit stresses imposed, surrounding conditions and relative conformity of the equation curve and that of the observed data. The equation is expressed

$$y = Cx^{\frac{1}{a}} \quad \text{or} \quad y = C\sqrt[a]{x}$$

y = millionths of an inch per inch per one pound per square inch

x = days

Exceptions to this definition for y are taken for the torsional data, Nos. 110 and 111 of Table 1. Here y is the angle of turn in radians times 10^{-5} for the load torque and cross-section of specimen used.

The use of logarithmic paper simplifies the work of determining curve equations of this sort for the data as reported from various sources, because when the data are plotted the points will appear to follow a straight line; the point where this line cuts the unit time line is the value C, and the slope of this line as drawn on the logarithmic paper is the exponent $1/a$. Figure 1 shows the plotting of an average case. For the sake of comparison the curve of the equation derived and the curve through the points plotted are shown on ordinary coordinate paper. Another plotting is shown on Figure 8.

The formulas set up from these coefficient and root values, as may be gathered from Fig. 1, do not entirely fit the observed data all along. As a general observation, it may be said that the conformity is quite good from a day or two to about a year; though large variations from these times occur for the various tests. (See Table 1 for a listing of conformity ranges.) In many of the Ohio tests good conformity starts at as short times as 15 minutes, and many of the California

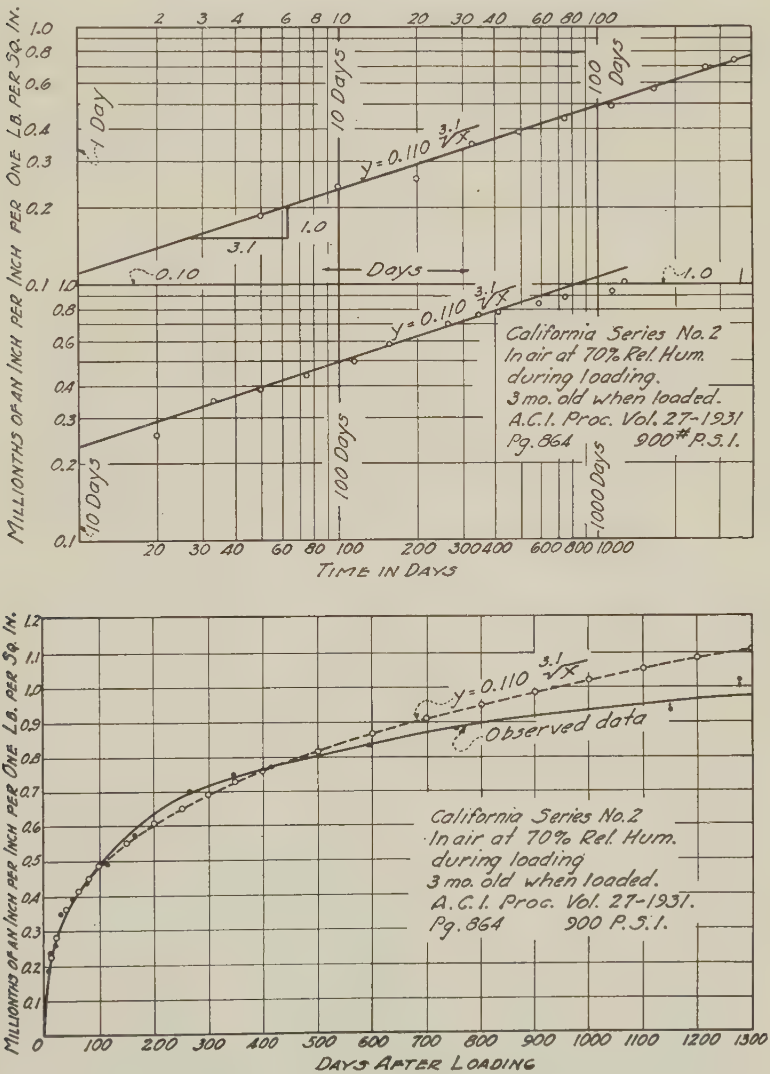

Fig. 1

TABLE 1—EXHIBIT OF DATA FOR PLASTIC FLOW TESTS WITH CURVE VALUES

Specimen Number	By Whom Tested and Where	Unit Stresses		Age at Loading Days	Conditions While Loaded		Cement Content By Weight		Kind of Aggregate		Fineness Modulus			Mod. of Elas. at Loading	Curve Valves $y = C\sqrt[a]{x}$		Range of Observations in Days	Straight Part of Curve in Days	Remarks
		p.s.i.	% Ult.		Rel. Hum. %	Temp. °F.	Mix	C/W	Coarse	Fine	Coarse	Fine	Mixed		C	a			
1	H and C O. S. U.	1383	33	28	20-50	70	1:2.8	?	Cols. L. S.	Cols. L. S.	6.88	3.54	5.70	4.15	0.087	2.26	0.10-60	0.10-60	
2	"	850	34	492	30-50	70	1:7.0	1.30	"	"	7.52	2.82	5.64	3.55	0.005	1.46	0.10-38	0.10-30	*
3	"	627	33	28	20-40	"	1:7.2	?	"	"	6.88	3.54	5.77	2.60	0.157	2.22	0.10-65	0.90-30	
4	"	2010	33	28	20-50	"	1:1.5	?	"	"	None	"	3.54	3.25	0.144	2.65	0.20-61	0.4-61	
5	"	1072	33	28	20-50	"	1:3.0	?	"	"	"	"	"	2.50	0.203	2.72	0.10-61	0.1-40	
6	"	874	33	28	20-50	"	1:4.8	?	"	"	6.88	"	5.77	"	0.150	2.47	0.10-102	1.0-60	
7	M and M O. S. U.	854	33	7	20-50	60	1:7.0	1.3	"	"	7.52	2.82	5.64	2.84	0.170	2.48	0.02-150	0.02-60	
8	"	876	34	14	"	"	"	"	"	"	"	"	"	"	0.172	2.86	0.02-143	0.10-143	
9	"	"	34	28	"	"	"	"	"	"	"	"	"	"	0.150	"	0.03-130	0.03-100	*
10	"	825	32	61	30-60	70	"	"	"	"	"	"	"	"	0.088	2.53	0.02-96	0.7-70	
11	"	876	34	120	"	75	"	"	"	"	"	"	"	"	0.078	2.35	0.07-37	0.07-37	
12	A and S O. S. U.	725	17.5	35	24-45	80	?	?	Gunite	Nat. S.	—	?	?	3.30	0.220	2.06	1.00-20	1.0-20	†
13	K and S O. S. U.	400	9.1	21	15-50	78	1:3.8	1.83	Silica Gravel	Silica	6.18	3.47	4.87	6.00	0.080	2.50	0.20-66	1.0-66	‡
14	"	800	18.3	21	"	"	"	"	"	"	"	"	"	"	0.110	3.50	0.02-66	0.02-66	
15	"	1200	27.5	21	"	"	"	"	"	"	"	"	"	"	0.093	3.20	0.05-66	0.05-66	
16	"	1600	36.6	21	"	"	"	"	"	"	"	"	"	"	0.085	2.80	0.10-66	0.10-66	
17	J and E O. S. U.	850	37.1	31	"	"	1:5.5	1.5	Col. L. S.	Col. L. S.	7.04	3.21	5.76	2.90	0.080	2.10	1.00-80	6.0-80	**
18	"	"	21.7	31	"	"	"	1.4	Silica Gravel	Silica	6.14	3.14	5.14	5.80	0.060	"	1.00-80	3.0-80	**
19	"	"	13.0	31	"	"	1:1.8	1.73	Col. L. S.	Col. L. S.	7.04	3.21	5.12	5.60	0.097	5.00	1.00-80	4.0-60	**
20	"	"	15	31	"	"	1:1.8	3.12	Silica Gravel	Silica	6.14	3.14	4.64	6.20	0.090	5.10	1.00-80	5.0-80	**
21	F and DiC O. S. U.	782	24	7	"	75	1:4.5	1.4	Gravel	Nat. S.	7.34	2.42	5.64	2.70	0.244	3.20	0.01-301	5.0-70	
22	"	"	25	7	"	"	1:3.7	?	Slag	Slag	6.59	3.00	4.63	4.23	? 0.143	? 3.10	0.03-312	16-100	
23	"	"	28	7	"	75	1:3.6	?	Haydite Cel- Sealed	Haydite Cel- Sealed	7.66	2.93	5.03	1.66	0.220	3.30	0.01-301	10-301	
24	"	"	44	7	"	"	1:4.1	?	"	"	6.44 7.03	2.04	4.43	1.28	0.300 0.570	2.55 8.70	0.01-331	0 to 10 0 to 331	
25	"	"	36	7	"	"	1:3.0	?	"	"	6.44 7.03	"	5.17	1.42	0.240 0.570	2.10 8.70	0.004-275	0 to 10 10 to 275	
26	L and S O. S. U.	739	24	28	60	"	1:6.0	1.15	L. S. & Gravel	L. S. & Nat. S.	—	—	—	3.12	0.132	2.60	0.005-597	0.01-100	
27	"	789	25	28	"	"	"	"	Gravel	Nat. S.	—	—	—	2.74	0.200	2.90	0.007-67	0.1-67	

28	R and K O. S. U.	750	62	28	15-50	75	1:5.5	1.50	Glass	Glass	7.09	3.58	5.80	1.75	0.210	2.60	0.003-317	Break at 35 da.	††
29	"	"	46	28	"	"	1:5.4	"	Ground Glass	"	7.08	"	"	2.30	0.160	2.30	0.05-315	Poor Curve	‡‡
30	"	"	28	28	"	"	1:6.1	"	Bedford L. S.	Bedford L. S.	"	"	5.60	2.25	0.110	3.30	0.03-313	Break at 25 da.	††
31	Davis, Calif.	640	18.5	213	78	70	1:4	1.85	Sand-Stone	Nat. S.	7.80	3.62	6.43	4.20	0.050	3.82	5-140	5-140	***
32	"	"	31	"	"	"	"	1.41	—	"	—	"	3.62	4.00	0.054	3.84	2-140	2-140	***
33	"	"	28	"	"	"	1:7	1.49	Sand-Stone	"	7.80	"	6.43	2.70	0.105	3.78	2-140	2-140	***
34	"	"	66	"	"	"	"	1.06	—	"	—	"	3.62	2.70	0.122	2.84	2-140	2-140	
35	"	600	28	92	Water	"	1:5	1.45	Granite	Granite	7.63	3.27	5.03	2.90	0.040	3.28	7-1273	20-1273	
36	"	900	40	"	"	"	"	"	"	"	"	"	"	2.70	0.062	3.84	6-1273	6-1273	†††
37	"	1200	53	"	"	"	"	"	"	"	"	"	"	2.40	0.094	4.93	5-1273	5-1273	†††
38	"	300	13	28	"	"	"	"	"	"	"	"	"	2.85	0.057	2.94	6-1360	6-200	
39	"	600	26	"	"	"	"	"	"	"	"	"	"	1.43	0.150	5.10	6-1360	6-600	
40	"	900	40	"	"	"	"	"	"	"	"	"	"	1.22	0.228	6.40	7-1360	7-600	
41	"	600	26	92	70	"	"	"	"	"	"	"	"	3.15	0.122	3.36	5-1278	5-1278	†††
42	"	900	40	"	"	"	"	"	"	"	"	"	"	2.60	0.110	3.10	5-1278	5-400	
43	"	1200	53	"	"	"	"	"	"	"	"	"	"	2.36	0.152	3.55	5-1278	5-1278	†††
44	"	300	13	28	"	"	"	"	"	"	"	"	"	3.44	0.242	4.40	8-1352	40-1352	
45	"	600	26	"	"	"	"	"	"	"	"	"	"	2.13	0.270	4.89	7-1352	45-1352	
46	"	900	40	"	"	"	"	"	"	"	"	"	"	1.47	0.350	5.65	7-1360	50-1360	
47	"	300	13	7	Water	"	"	"	"	"	"	"	"	1.69	0.240	8.25	5-1293	10-1293	
48	"	600	26	7	"	"	"	"	"	"	"	"	"	0.945	0.570	12.45	5-1292	5-1292	†††
49	"	800	—	28	Water	70	1:5.7	1.69	Gravel	Nat. S.	6.74	3.48	5.60	2.90	0.080	4.50	5-672	5-672	***
50	"	"	—	"	100	"	"	"	"	"	"	"	"	"	0.093	4.62	1-686	35-686	
51	"	"	—	"	70	"	"	"	"	"	"	"	"	"	0.124	3.37	1-633	1-633	†††
52	"	"	—	"	50	"	"	"	"	"	"	"	"	"	0.160	2.50	5-659	5-93	
53	"	"	—	"	"	"	"	"	Lime-Stone	Lime-Stone	—	—	"	3.50	0.054	2.32	5-628	5-250	
54	"	"	—	"	"	"	"	"	Quartz	Quartz	—	–	"	3.65	0.067	2.44	14-648	14-580	***
55	"	"	—	"	"	"	"	"	Granite	Granite	—	—	"	3.80	0.125	5.04	30-672	60-580	***
56	"	"	—	"	"	"	"	"	Gravel	Mixed	6.74	3.48	"	2.80	0.207	3.80	7-659	26-659	
57	"	"	—	"	"	"	"	"	Basalt	Basalt	—	—	"	3.30	0.170	3.20	27-673	Curved all along	
58	"	"	—	"	"	"	"	"	Sand-Stone	Sand-Stone	—	—	"	2.75	0.106	2.07	3-680	5-100	
59	"	"	—	60	"	"	1:5	1.76	"	Nat. S.	—	—	5.22	4.00	0.060	2.40	2-538	2-538	†††
60	"	"	16	28	Water	"	1:3.6	2.38	Gravel	"	6.39	2.15	5.70	4.14	0.044	3.52	2-333	5-220	
61	"	"	"	"	50	"	"	"	"	"	"	"	"	3.33	0.138	3.85	1-330	3-330	
62	"	"	25	"	Water	"	1:6.3	1.52	"	"	6.30	"	5.55	4.00	0.088	6.00	1-300	1-300	†††
63	"	"	"	"	50	"	"	"	"	"	"	"	"	3.64	0.143	3.66	2-308	6-220	
64	Glanville, England	200	?	"	Air	65	1:4.6	1.43	"	"	6.32	3.09	5.24	?	0.107	3.40	7-90	24-90	
65	"	400	?	"	"	"	"	"	"	"	"	"	"	?	0.107	3.20	7-90	7-90	†††
66	"	600	?	"	"	"	"	"	"	"	"	"	"	?	0.118	3.10	7-90	24-90	

(Table 1 continued p. 154)

TABLE 1—*Continued*

Specimen Number	By Whom Tested and Where	Unit Stresses		Age at Loading Days	Conditions While Loaded		Cement Content By Weight		Kind of Aggregate		Fineness Modulus			Mod. of Elas. at Loading	Curve Valves $y = C\sqrt[a]{x}$		Range of Observations in Days	Straight Part of Curve in Days	Remarks
		p.s.i.	% Ult.		Rel. Hum. %	Temp. °F.	Mix	C/W	Coarse	Fine	Coarse	Fine	Mixed		C	a			
67	Glanville, England	800	?	28	Air	65	1:4.6	1.43	Gravel	Nat. S.	6.32	3.09	5.24	?	0.192	3.90	7-90	24-90	
68	"	600	27.4	15	"	"	"	"	"	"	"	"	"	2.86	0.130	2.80	7-180	14-90	
69	"	"	"	28	"	"	"	"	"	"	"	"	"	3.43	0.098	2.40	7-330	7-90	
70	"	"	"	90	"	"	"	"	"	"	"	"	"	3.75	0.072	3.00	7-90	24-90	
71	"	"	13.3	28	"	"	1:2.3	2.50	"	"	"	"	"	5.26	0.076	3.80	7-330	24-330	
72	"	"	27.4	"	"	"	1:4.6	1.43	"	"	"	"	"	3.43	0.113	2.70	7-330	7-150	
73	"	"	47.4	"	"	"	1:6.9	1.18	"	"	"	"	"	2.86	0.370	4.20	7-330	20-330	‡‡‡
74	"	300	23.2	"	"	"	1:4.6	1.11	"	"	"	"	"	2.10	0.100	2.60	7-150	7-150	
75	"	600	27.4	"	"	"	"	1.43	"	"	"	"	"	3.43	0.110	2.60	7-150	7-150	
76	"	"	27.5	"	"	"	"	"	"	"	"	"	"	3.43	0.098	2.40	7-90	7-90	†††
77	"	"	29	"	Water	"	"	"	"	"	"	"	"	3.64		Bad	Curve		
							Rapid Hardening or High Early Strength Cements												
78	Glanville, England	1871	47	28	Air	65	1:3.5	1.67	Gravel	Nat. S.	6.32	3.09	5.24	3.72	0.420	2.10	5 Secs. to 60 secs.	5s-60s	†††
79	"	1712	43	"	"	"	"	"	"	"	"	"	"	"	0.164	2.60	"	"	
80	"	1554	39	"	"	"	"	"	"	"	"	"	"	"	0.760	1.70	"	"	
81	"	1396	35	"	"	"	"	"	"	"	"	"	"	"	0.260	2.10	"	"	***
82	"	1273	32	"	"	"	"	"	"	"	"	"	"	"	1.350	1.20	"	"	
83	"	1078	27	"	"	"	"	"	"	"	"	"	"	"	0.220	2.00	"	"	***
84	"	300	7.5	"	"	"	"	"	"	"	"	"	"	"	0.059	2.20	5-177	10-177	
85	"	600	15	"	"	"	"	"	"	"	"	"	"	"	0.059	2.50	"	10-177	
86	"	900	22.5	"	"	"	"	"	"	"	"	"	"	"	0.048	2.40	"	10-177	
87	"	1200	30	"	"	"	"	"	"	"	"	"	"	"	0.059	2.60	"	5-120	
88	"	1500	37	"	"	"	"	"	"	"	"	"	"	"	0.060	2.60	"	5-120	
89	"	600	22	7	"	"	1:4.6	1.43	"	"	"	"	"	3.87	0.096	3.30	7-180	7-90	
90	"	"	"	14	"	"	"	"	"	"	"	"	"	4.00	0.103	7.90	"	24-180	
91	"	"	"	28	"	"	"	"	"	"	"	"	"	4.11	0.053	3.20	7-330	7-330	
92	"	"	"	90	"	"	"	"	"	"	"	"	"	4.41	0.026	2.90	7-90	7-90	
93	"	"	"	28	"	"	"	"	"	"	"	"	"	4.11	0.064	3.90	"	24-90	
94	"	"	20.3	29	Water	"	"	"	"	"	"	"	"	3.77	0.055	4.30	"	24-90	‡‡‡

									Aluminous Cements										
95	Glanville, England	600	12.9	1	Air	65	1:4.6	1.43	Gravel	Nat. S.	6.32	3.09	5.24	4.29	0.100	3.40	7-180	7-180	
96	"	"	"	7	"	"	"	"	"	"	"	"	"	4.62	0.049	2.50	"	7-180	
97	"	"	"	28	"	"	"	"	"	"	"	"	"	5.56	0.036	2.60	7-330	14-230	
98	"	"	"	90	"	"	"	"	"	"	"	"	"	5.21	0.0225	2.20	7-90	14-90	
99	"	"	13	28	"	"	"	"	"	"	"	"	"	5.55	0.035	2.50	"	24-90	
100	"	"	18	29	Water	"	"	"	"	"	"	"	"	4.54	0.094	4.00	"	"	
									Plastic Recovery—Portland Cement Concrete										
101	H and C O. S. U.	874	33	93	20-50	70	1:4.8	?	Col. L. S.	Col. L. S.	6.88	3.54	5.77	2.50	0.053	4.60	0.3-35.2	0.3-35.2	x
102	Davis, Calif.	800	16	365	Water	"	1:3.6	2.38	Gravel	Nat. S.	6.39	2.15	5.70	4.14	0.187	4.10	0.2-4.3	0.2-4.3	†††
103	"	"	"	"	50	"	"	"	"	"	"	"	"	3.33	0.240	5.90	0.7-5.0	0.7-5.0	†††
104	"	"	?	273	"	"	1:5.7	1.76	"	"	6.72	2.32	4.95	3.30		Bad	Curve		
105	"	"	?	"	Water	"	"	"	"	"	"	"	"	3.40		"	"		
									Celluloid as a Medium for Photo-Elastic Investigation—Compression										
106	E & McQ Wash.	1000	—	—	—	—	—	—	—	—	—	—	—	—	0.620	2.0	2 Min. to 150 Min.	Full Time	†††
107	"	2000	—	—	—	—	—	—	—	—	—	—	—	—	0.785	2.9	"	"	†††
108	"	3000	—	—	—	—	—	—	—	—	—	—	—	—	0.976	3.0	"	6 Min. to 150 Min.	
109	"	4000	—	—	—	—	—	—	—	—	—	—	—	—	1.600	2.6	"	Full Time	†††
									Torsion Specimens From Plot, Page 248, *Proceedings*, A. S. C. E., Feb. 1935										
110	Anderson, Illinois	y is radians x 10^{-5} for unknown torque and cross section													1.80	4.7	1-105	1-105	
111	"	" " " " " " "													1.50	6.7	"	7-105	

*Nos. 2 and 9 are tests run on the same specimen.
†No. 12 curve breaks at 1 day and 20 days. Below 1 day the root is 1.1 and over 20 days the root is 16.0. The power formula does not fit Gunite.
‡No. 13 curve showed an upturn below 1 day. Root 1.3.
**Nos. 17, 18, 19 and 20 showed the usual downturn.
††Sudden changes in the slopes to a flatter slope, higher root.
‡‡A slip occurred at 10 days steepening the slope to double or more. After 30 days the old slope was resumed which began to flatten at 60 days.
***The observed points were erratic, jumping above and below according to no apparent rule.
†††The conformity of these was excellent and complete.
‡‡‡A general curve on the log sheet, ends down.
xSame specimen as No. 6.

tests show conformity carrying through to as much as three or more years. The Ohio tests do not carry through for long times, and the California test data at hand seldom start at less than two days. When non-conformity obtains it usually shows up for the long times, as shown on Fig. 1 as a falling away from the straight line on the log plot, which indicates an increase in both coefficient and root. The increase in the coefficient comes about by reason of the extrapolation back to the one day line and results largely from the reduced slope. Non-conformity at the short times, when it does occur, again shows up as a falling away from the straight line as an increase in the coefficient and a decrease in the root. Here again the coefficient is determined by the change in the slope, and since for time less than one day, the extrapolation is on the other side, the decrease in the root (increase in slope) produces again an increase in the coefficient. In some cases a line drawn through the points plotted on the log paper is a curve throughout, the ends bending downward, indicating possibly some other law than the power expression. As these cases usually obtained for odd conditions of aggregates or mixtures and for only a few of the tests, they were considered as exceptions and not sufficiently indicative to warrant an attempt to find a new law.

A series of tests in England (15), designed to show how plastic flow influences the test determination of the modulus of elasticity, throws some interesting light on the conditions at the short time end. A rather rich rapid hardening cement concrete was tested for plastic flow in a period of 60 seconds and readings were taken at 15 seconds intervals. Six tests were made for different loadings. The averages for the coefficients and roots are 0.529 and 1.95 respectively. These appear on Table 1 as Nos. 78 to 83, inclusive. It will be noticed that they are all heavily loaded, three of them being overloaded. This may be compared to a leaner concrete of the same type of cement which produced the values of 0.064 and 3.9 respectively, No. 93, Table 1. The steeper slope at the short time end is here illustrated in these six tests. The large difference between the coefficients should not be taken too seriously since the extrapolation for determining them from the 60 seconds tests is very great, to a point away from the 60 second range of 1440 times the time at the end of the range. It is of interest to note that one of the 60 seconds tests, No. 79, gave 0.164 and 2.6 for C and a, which compare favorably with the general average values.

In view of the number of Ohio tests, where good conformity was had, to as low as 0.02 of a day or 30 minutes (7, 9, 11, and 14 of Table 1), and because of the small effect, from a practical standpoint, to the

engineer of the amount of plastic flow at these short times, and in spite of the English tests, it is believed that the values as given in Table 1 may safely be used without correction for the short times up to the point where the long time drop-off begins to take place. This long time drop-off, however, cannot be ignored.

For the longer times the exponential formula here proposed does not conform in general to the results of the long time tests made at the University of California nor does it conform to Thomas' limiting value theory; but the long time tests do agree fairly well with this limiting value theory.

Some transition correction is needed to apply to the power expression between the drop-off point and the time of limiting value. An examination of the data on Table 1 gives some clue as to when this drop-off should be considered to start.

The Ohio tests cannot be taken strongly into account when considering the drop-off points because there is a reason why the drop-off points should occur early. These tests were carried on under ordinary room conditions and usually started about January first when the interior humidities were quite low (as low as 10 per cent at times) because of the artificial heating. After about two to three months, warmer weather and reduction in the heating caused the inside humidities to rise. This rise in humidity had a tendency to reduce the rate of plastic flow and cause a drop-off. Out of 13 tests where a drop-off was noticed in the time ranges of observation, the time of drop-off occurred at 60 days. Ten tests, showing straight lines or conformity to the ends of the ranges to times longer than 60 days, averaged 77 days.

The other tests where the humidities were controlled, particularly those carried on in California, offered good data for making these observations. Out of 16 tests where a drop-off was noted the drop-off time averaged 280 days. The times of 19 more, where conformity continued throughout, averaged 916 days. Assuming that the drop-off was just about to occur at the ends of the times for these 19 tests, the average drop-off time occurred at 540 days or about one and one-half years. The shortest time was 90 days and the longest something over 1352 days. See Nos. 44 and 45, Table 1.

Mr. Thomas indicates that the limit of plastic flow for specimens loaded at 28 days may be taken as 27 per cent more than the plastic flow at one year. For shorter time loadings the increase is less, about 17 per cent for seven day loadings, and for longer time loadings the increase is more, about 40 per cent for three months loadings. The long time tests at the University of California bear out this conclusion

fairly well. A 27 per cent increase over that at one year for ordinary concretes using ordinary aggregates, loaded in air at 28 days, C and a being 0.128 and 2.9 respectively, occurs at 733 days or at about two years.

It is suggested that when calculations are to be made for plastic flows for times greater than one year, the plastic flows calculated from the power formula be reduced according to a system such as the following:

Follow the power expression up to one year. From one year to two years, to the power formula value for one year, for 28 day loading, add 70 per cent of the excess over that at one year. From two years to three years, add to this reduced value at two years 40 per cent of the power formula value greater than at two years. From three years to four years, add to this reduced value at three years 20 per cent of the power formula value greater than at three years. From four years to five years, add to this reduced value at four years 10 per cent of the power formula value greater than at four years. Consider the limiting value to have occurred at five years. For seven day loadings change these percentages to 50, 25, 12, and 6 per cent and for three month loading to 90, 75, 50, and 25 per cents respectively.

A shorter and more practical, and probably sufficiently accurate way of handling this is to consider the exponential curve useful only to one year and for times greater than one year to jump to Thomas' limiting value.

CLASSIFICATION AND ANALYSIS OF DATA

To study and present these data so that they may be of service to computers and designers, the following more or less arbitrary divisions were made (See Table 2).

A division was made between portland cement concretes working in air and those working in water; between concretes in buildings, superstructures for bridges, and other types of above ground construction, where constant contact with water or one hundred per cent relative humidity was unlikely, in contrast to those for hydraulic structures as for water purification, sewage disposal, and other like structures and underground structures where constant 100 per cent moisture conditions prevail. All concretes tested under relative humidity conditions of 70 per cent or less were included in the first division and those under water or in conditions of 100 per cent humidity or nearly that were included in the latter.

Another division was made between ordinary strength concretes and rich or high strength concretes. Concretes made with aggregates whose combined fineness moduli ranged between 5.0 and 6.0 and whose cement-water ratios by weight ranged between 1.2 and 1.6 were included in the former class. Those with cement-water ratios over 1.6 and fineness moduli between 3.0 and 6.0 were included in the latter.

TABLE 2—RESUME OF POWER CURVE DATA

Portland Cement Concretes

Conditions of Concrete

Strength	Aggregates	Surroundings	Age at Loading	Equation Values C	Equation Values a	Average of
Ordinary	Ordinary	In Air	7 days	0.207	2.8	2
"	"	" "	14 days	0.151	2.8	2
"	"	" "	28 days	0.128	2.9	16
"	"	" "	2 months	0.088	2.5	1
"	"	" "	3 months	0.072	3.0	1
"	"	" "	4 months	0.078	2.3	1
"	"	" "	16 months	0.005	1.5	1
"	"	In Water	28 days	0.088	6.0	1
Rich	"	In Air	28 days	0.129	3.2	10
"	"	In Water	28 days	0.072	4.2	3
Ordinary	Granite	In Air	28 days	0.287	5.0	3
"	"	" "	3 months	0.128	3.3	3
"	"	In Water	7 days	0.405	10.0	2
"	"	" "	28 days	0.145	4.8	3
"	"	" "	3 months	0.065	4.0	3
Rich	"	In Air	28 days	0.125	5.0	1
Ordinary	Sandstone	" "	7 days	0.105	3.8	1
Rich	"	" "	28 days	0.132	2.6	1
"	"	" "	2 months	0.060	2.4	1
"	"	" "	7 months	0.050	3.8	1
Ordinary	Silica	" "	28 days	0.086	2.8	5
Rich	"	" "	28 days	0.078	3.8	2
"	Basalt	" "	28 days	0.170	3.2	1
Lean, overloaded	Ordinary	" "	28 days	0.370	4.2	1
Ordinary, overloaded	Sandstone	" "	7 months	0.122	2.8	1
" "	Granite	" "	3 months	0.152	3.6	1
" "	"	In Water	3 months	0.094	4.7	1
" "	Ground glass Coarse Crushed glass Fine	In Air	28 days	0.160	2.3	1
Lean	Ordinary	In Air	28 days	0.100	2.6	1

Rapid Hardening Cement Concrete

Conditions of Concrete

Strength	Aggregates	Surroundings	Age at Loading	Equation Values C	Equation Values a	Average of
Ordinary	Ordinary	In Air	7 days	0.096	3.3	1
"	"	" "	14 days	0.103	3.3	1
"	"	" "	28 days	0.053	3.2	1
"	"	" "	3 months	0.026	2.9	1
"	"	In Water	28 days	0.055	4.3	1
Rich	"	In Air	28 days	0.057	2.5	5

Aluminous Cement Concrete

Conditions of Concrete

Strength	Aggregates	Surroundings	Age at Loading	Equation Values C	Equation Values a	Average of
Ordinary	Ordinary	In Air	7 days	0.100	3.4	1
"	"	" "	14 days	0.049	2.5	1
"	"	" "	28 days	0.036	2.6	2
"	"	" "	3 months	0.023	2.2	1
"	"	In Water	28 days	0.094	4.0	1

A division was made between concretes containing ordinary aggregates, such as gravel and natural sand and crushed limestone, both coarse and fine, or crushed limestone with natural sand; and those containing special aggregates such as silica, crushed or in the form of pebbles, granite, sandstone, basalt, slag, or especially manufactured aggregates such as Haydite, etc.

Especially lean concretes, those having a cement-water ratio of less than 1.2 or overloaded concretes, those having a unit stress to ultimate stress ratio of more than 0.4 were kept in classes by themselves. High early strength cement concretes and aluminous cement concretes were also kept in classes by themselves.

The most important set of data in Table 2 is that for ordinary strength concrete made of ordinary aggregate and loaded in air at 28 days. The averages of C and a for 16 tests are 0.128 and 2.9 respectively. The values for C ranged from 0.080 to 0.200 and those for a from 2.10 to 3.66. Another group of sufficient number to be considered is that of the rich concrete using ordinary aggregates and loaded in air at 28 days. The average values for C and a of 10 tests are 0.129 and 3.2 respectively. The ranges are 0.054 to 0.207 for C and 2.26 to 5.00 for a. If that one whose a value is 5.00 (No. 19 of Table 1) which had generally poor conformation of curve to points observed be left out, the averages would be 0.133 and 3.03 respectively for C and a, and the ranges would be 0.054 to 0.207 and 2.26 to 3.85. If these two are grouped together the averages will be 0.130 for C and 3.14 for a which is very close to the cubic parabola

$$y = 0.13\sqrt[3]{x}$$

From the data of these 25 tests the theory of probability would indicate that the chances are 25 to 1 that all possible variations will be within 0.079 or 61 per cent for C and 1.16 or 39 per cent for a.

EFFECT OF AGE AT LOADING

That the plastic flow is materially influenced by the age at loading has been shown in reports of investigations from all sources. Table 2 shows this influence in a general way. Observations for quantitative comparisons of the results of Table 2 are somewhat befogged by the variations of the two values C and a. Since a change in the root primarily changes the shape of the curve, direct comparisons cannot be made on the coefficient C except when the roots are the same. It is impossible to transform whole curves to a common root, but it is possible to take points on a set of curves, say 28 days, and transform them to the same root insofar as plastic flow at 28 days is concerned. It is then possible to make direct quantitative comparisons by way of the coefficients. The equation for this transformation is

$$y = C_1\sqrt[a1]{x} = C_2\sqrt[a2]{x} \qquad \text{where}$$

y = the plastic flow at the common time in days
C_1 = the observed coefficient
a_1 = the observed root
a_2 = the root to which the transformation is to be made
C_2 = the unknown coefficient to be solved for

By this method a substitute curve expression is found which will intersect the original curve at the common time x days. Other points of days may be selected and transformation values solved for the coefficients. If there is a large difference between the observed root and the one to which the transformation is made there will be a

correspondingly large variation in the coefficients found for the different points of days selected.

This procedure was carried out for the different ages at loading for the "Ordinary, Ordinary, In Air" tier of data, Table 2, for portland cement for the points of days of 2, 28, 90, and 365 days for transformation to the root 3.0. The curve through the coefficient averages as ordinates and ages at loading for abscissas had a shape similar to the general plot for plastic flow, though the curvature was in the other direction. When this analysis was attempted upon log paper it appeared that a exponential power curve could be drawn for this relation. The variation of the C values against age at loading could be expressed thus:

$C = 0.500\ \alpha^{-2.5}$ where

$\alpha =$ the age at loading.

Since this expression was worked up for values transformed to a common root 3.0 for the general exponential expression, it is necessary to transform back to the root desired for any given expression.

Table II offers two other sets of data from which deductions may be made for the effect of age at loading, "Ordinary Granite, In Water" and "Rich, Sandstone, In Air." The formula for the former is

$C = 0.780\ \alpha^{-1.62}$

and for the latter

$C = 1.50\ \alpha^{-1.50}$

These two are also worked out on the basis of the root 3.0.

EFFECT OF CURING CONDITIONS

Table 2 shows in five comparisons that concretes in water do not in general flow as much as those in air, though the flow is more rapid immediately after loading. In all cases the coefficient for the concretes in water is lower and in all but one case the root is higher. Three of these comparisons have been studied in some detail and are given in Table 3. These three were selected because not less than three tests were averaged to make up each value.

From a study of this table it can be said that the plastic flow of concretes in water is about half that in air, less than that when short time flows are considered and greater when long time flows are considered.

DIFFERENT KINDS OF AGGREGATES

The effect on plastic flow of different kinds of aggregates is not so clearly defined as the two effects already discussed. It is not under-

TABLE 3—RATIOS OF PLASTIC FLOWS. CONCRETES IN WATER TO CONCRETES IN AIR

Time, Days	Rich Ordinary Agg. Concrete Loaded at 28 Days	Ordinary Granite Concrete Loaded at 28 Days	Ordinary Granite Concrete Loaded at 3 Months	Average
0.02	0.746	0.490	0.625	0.620
0.5	0.586	0.502	0.530	0.539
2.0	0.532	0.507	0.490	0.510
10.0	0.470	0.515	0.450	0.478
28.0	0.434	0.520	0.426	0.460
90.0	0.398	0.526	0.400	0.441
365.0	0.358	0.532	0.372	0.421
			Average	0.496

stood how or in what way different aggregates cause these variations. That it is due to plastic flow in the aggregate particles themselves can safely be ruled out (See discussion of Bibliography No. 13) excepting possibly in the case of some of the very special porous aggregates. There remain the shape of the particle, and its surface characteristics. The surface characteristics that might be considered are hardness, smoothness, density, or porousness, cleavage (planar for crystal surfaces, as in granite or rounded as in pebbles) and adhesiveness. Table 2 shows that a difference exists, the problem remaining is to determine why.

To make proper comparisons it is again apparent that the coefficients in Table 2 must be brought down to a common root for a number of time points. The values for those available for this comparison were transformed to a common root 3.0 for 2, 28, 90, and 365 days, and the averages were taken. From these it appears that granite concrete shows a plastic flow 1.38 times as much as for ordinary aggregate; sandstone concrete, 0.85 times as much; silica aggregate 0.65 times, and basalt aggregate 1.32 times.

The influence of different aggregates is seen to be not nearly as definite as the effect of age at loading and the effect of water curing. The data at hand are such that even general statements to the effect that more plastic flow may be had from granite aggregate can hardly be relied upon.

EFFECT OF UNIT STRESS

The exponential formula presented in this paper expresses the plastic flow in terms of millionths of an inch per inch for a unit stress of one pound per square inch on the assumption that the plastic flow is proportional to the stress applied. Table 4 shows how this assumption is borne out by the data at hand from California, England, and Ohio for concretes made up of a variety of aggregate materials using portland cement and for one set for concretes using high early strength

TABLE 4—EFFECT OF UNIT STRESS ON PLASTIC FLOW

Group	Av. Values a and C		No. Table 1	Unit Stress	Per Cent of Ult.	a and C		Av. Transformed a and C		Per Cent of Av.
Ord. Ord.			64	200	?	3.4	0.107	3.4	0.107	82
Air	3.4	0.131	65	400	?	3.2	0.107	3.4	0.114	88
28 days			66	600	?	3.1	0.118	3.4	0.131	101
Portland Cement			67	800	?	3.9	0.192	3.4	0.168	129
Rich Silica			13	400	9	2.5	0.080	3.0	0.1025	110
Air	3.0	0.092	14	800	18	3.5	0.110	3.0	0.093	100
21 days			15	1200	28	3.2	0.093	3.0	0.0865	93
Portland Cement			16	1600	37	2.8	0.085	3.0	0.092	98
Ord. Granite			44	300	13	4.4	0.242	5.0	0.267	93
Air (P. C.)	5.0	0.287	45	600	26	4.89	0.270	5.0	0.274	95
28 days			46	900	40	5.65	0.350	5.0	0.322	112
Ord. Granite			38	300	13	2.94	0.057	4.8	0.095	66
Water (P. C.)	4.8	0.145	39	600	26	5.10	0.150	4.8	0.143	100
28 days			40	900	40	6.40	0.228	4.8	0.190	133
Ord. Granite			41	600	26	3.36	0.122	3.3	0.119	94
Air (P. C.)	3.3	0.128	42	900	40	3.10	0.110	3.3	0.118	94
3 months			43	1200	53	3.55	0.152	3.3	0.141	112
Ord. Granite			35	600	28	3.28	0.040	4.0	0.045	72
Water (P. C.)	4.0	0.065	36	900	40	3.84	0.062	4.0	0.063	100
3 months			37	1200	53	4.93	0.094	4.0	0.080	127
H. E.			84	300	7.5	2.20	0.059	2.46	0.0705	123
Cement			85	600	15	2.50	0.059	2.46	0.0575	100
Ord. Ord.	2.46	0.057	86	900	22.5	2.40	0.048	2.46	0.050	87
Air			87	1200	30	2.60	0.058	2.46	0.0535	93
28 days			88	1500	37.5	2.60	0.060	2.46	0.0555	97

cement called "rapid hardening cement" by the English investigators. The values "Av. Transformed *a* and *C*" are averages for transformations of *C* to the average *a* value for the group for 2, 28, 90, and 365 days as was done for other comparisons.

A general advance in the percentages is noticed in the granite concretes in the "Ord. Ord. Air 28 day" group. The silica and the high early strength cement concrete groups seem to show the reverse, or rather no general trend. The granite concretes in air show little change until the unit stress gets quite high, 40 and 53 per cent of the ultimate, which is hardly within the range of working stresses. The general advance for the "Ord. Ord. Air 28 day" group cannot be explained. The same may be said of the granite concretes in water, but it is noticed that these conditions are special. Whether or not the plastic flow is proportional to the stress for concretes in water can hardly be definitely stated from these few tests, but it is possible that this lack of proportionality is general.

HIGH EARLY STRENGTH CEMENTS

All the data at hand on high early strength or rapid hardening cements came from England. A comparison of all six groups, Table 2

with their corresponding groups among the portland cement concretes showed a reduction in plastic flow for the high early strength cement concretes. The average for all was 50 per cent. The highest ratio was that for "Rich, Ordinary, In Air, 28 days" which was 61 per cent. At three months the ratio was 53 per cent.

The effect of age at loading was much the same for rapid hardening as for ordinary cement; the expression works out roughly to be

$C = 0.26\,\alpha^{-2}$ which is between the general expression and that for rich sandstone in air.

The effect due to the concrete working in water was in the same direction but not so great as for the portland cements. The ratio of plastic flow in water to that in air is 0.80.

ALUMINOUS CEMENTS

The data for the aluminous cements also all came from England. Here again as for the high early cement concretes the plastic flow was not nearly as great as for the normal portland cement concretes. The average ratio for the specimens kept in air is 44 per cent. The smallest appeared at seven days and the largest at 90 days. The three ratios for seven days, 28 days, and 90 days are 28 per cent, 33 per cent, and 72 per cent.

The effect of age at loading was in the same direction as for normal portland cements or high early strength cements but not nearly so great. The expression is

$$C = 0.090\,\alpha^{-5.4}$$

The effect due to the concrete working in water was found to be opposite to that of the other two. The ratio of the plastic flow for the concrete in water to that in air is 1.18.

MECHANICS OF PLASTIC FLOW

Deformations in concrete other than elastic deformations particularly those due to shrinkage and plastic flow, present problems in mechanics the solutions of which are not now generally found in texts. Only a few attempts at solution have so far appeared in technical journals and research publications (Bibliography Nos. 10, 15, 16, and 17). In order to help remedy this deficiency a number of theoretical expressions and procedures will be developed and set forth in this paper. A set of tests will be described which confirms a portion of this theory to some extent.

The general field might be divided into plain concrete and reinforced concrete. A number of other divisions are necessary. Shrinkage and plastic flow are not alike in their action and should be dealt with separately. Plastic flow problems are divided into two main classes, those resulting from sustained load, and those from sustained strain such as constant deflection. Structural members, particularly those of reinforced concrete, are classified as columns or beams. It is possible that beams should be subdivided into those simply or singly reinforced and those carrying steel in compression as well as in tension, often spoken of as doubly reinforced beams.

Shrinkage of Columns

Shrinkage of plain concrete produces no difficult problems of stress distribution except where the externals, forms and stresses, of other members are affected.

Shrinkage stresses in symmetrically reinforced members may be solved very easily.

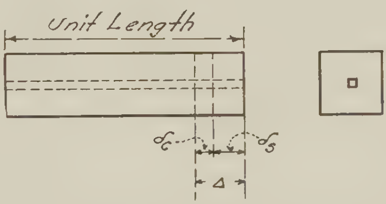

FIG. 2

Given a unit length of a symmetrically reinforced member, Fig. 2. The plain concrete shrinkage is represented by Δ. The reinforced member tends to shrink Δ but the steel, not being affected by humidity and drying out, resists this shrinkage and the actual shrinkage of the reinforced member is δ_s. It is assumed that the steel is perfectly bonded to the concrete. The concrete is then elongated δ_c and the steel is compressed or shortened δ_s. Since no external forces are considered, the total stress on the steel equals that on the concrete and will be designated by P.

$$E_s = \frac{P/A_s}{\delta_s} \qquad \text{and} \qquad E_c = \frac{P/A_c}{\delta_c}$$

$$\text{and} \qquad \delta_s + \delta_c = \Delta$$

from which $$P = E_s \Delta A \frac{p(1-p)}{1+p(n-1)}$$

From this formula it is easy to compute the unit stresses in either the steel or the concrete.

This expression is worked out on the basis of perfect bond between the steel and the concrete and for any unit length of a symmetrically reinforced member. The value P and the unit stresses are therefore constant from end to end of the member and consequently there is no differential to produce bond stress. Now it is obvious that complete absence of bond stress is incompatible with the practical situation. The theoretical answer to this is, therefore that infinite bond resistance is demanded at the ends of a column or other member whether long or short. This condition again is impractical. The practical situation then is that a length of embedment is necessary at the ends to develop this bond requirement before the full value of P can obtain. This requirement means that slippage occurs at the extreme ends and the length of embedment necessary will depend upon the surface conditions of the steel and the ability of the concrete to bond heal after very slight slippages have occurred and upon the bond plastic flow property of the concrete.

Little has been published so far on tests for bond plastic flow but what has been published (13) and tests now in progress indicate that the property is worthy of investigation. Glanville (26) has made and reported tests on shrinkage stresses and shows in some cases this required length of embedment to be from three to 36 inches for ½ and ¾ inch round bars.

The value P when shrinkage occurs at early ages is materially reduced by the plastic flow of the concrete. So is the tendency for the concrete to crack. Shrinkage movements are not dependent upon age as are plastic flow movements so that the latter cannot always be depended upon to reduce the stresses. As the concrete grows older the tendency to crack is increased and probably a much greater length of end embedment is necessary. The full stresses due to shrinkage are then not so widely distributed over a member but the shrinkage actions apparently tend to interfere with and impair the bond resistance at regions of cracks and at the free ends.

Shrinkage Warping of Beams

In considering the bowing or warping of singly reinforced concrete beams the same procedure as that for the symmetrically reinforced member will be used with the addition of bending action and the general principle of area moments. See Fig. 3.

The presence of steel off the neutral axis of the section will cause the vertical straight line before bending to become an inclined straight line after bending and a couple will be formed whose arm is the dis-

tance between the neutral axis of the compound section and the center of gravity of the steel. The moment of the area of half of the M/EI diagram, involving this moment about the end will give the middle ordinate of the bow.

$$E_s = \frac{P/A_s}{\delta_s} \qquad E_c = \frac{P/bd}{\delta_c} \qquad M = \frac{f_b I_c}{r} \qquad = Pr$$

$$\delta_s = \frac{P}{E_s pbd} \qquad \delta_c = \frac{Pn}{E_s bd} \qquad \delta_b = \frac{Pnr^2}{E_s I_c}$$

$$\Delta = \delta_s + \delta_c + \delta_b$$

$$P = \frac{E_s \Delta}{\frac{1}{pbd} + \frac{n}{bd} + \frac{nr^2}{I_c}}$$

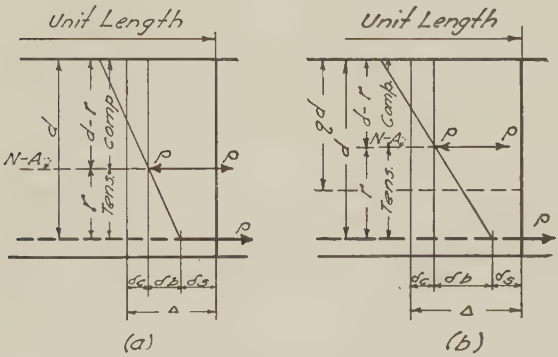

FIG. 3

Here the neutral axis for the entire concrete and transformed steel sections is r distant from the center of gravity of the steel. The moment of inertia I_c is also taken for the entire working cross-section.

Fig. 3 applies all along the length l of the beam and the moment M is constant. The moment diagrams is then a rectangle Ml. The length l should be selected not as the entire overall length but with the necessary end lengths of embedment in mind. The central deflection or bow is

$$D = \frac{Ml^2}{8E_c I_c} = \frac{Prl^2 n}{8E_s I_c}$$

This formula has been worked out upon the assumption that the entire depth d is working or that no cracks have penetrated beyond the steel, and also that cracks have entirely destroyed the value of the concrete below or outside of the steel. If the beam has been

loaded, so that it is known that cracks have been formed, which penetrate much farther than the steel, some change in this procedure must be made. It may be well to use the term "q" to represent a fraction of d so that qd represents that part of the depth d which is working. See Fig. 3b. It is only feasible to assume this value q to be constant for the full length of the beam and the basis for assuming a value for q is found in test data. Fig. 4 shows one of several tests made at Ohio State University from which such an assumption may be made.

The values for δ_s, δ_c, and δ_b are then:

$$\delta_s = \frac{P}{E_s pbd} \qquad \delta_c = \frac{Pn}{E_s bqd} \qquad \delta_b = \frac{Pnr^2}{E_s I_c}$$

It should be noted that r and I_c must be computed from the reduced section qd instead of from d.

Calculations for conditions involving compression reinforcement such as for doubly reinforced beams, arch rings, and columns are no more difficult. The couple arm in all cases is the distance from the neutral axis of the entire section to the center of gravity of *all* of the steel. The location of the neutral axis and the moment of inertia are computed from the total working area comprising the working area of the concrete, and all the steel. In transforming steel areas which are in a working area of concrete it is a bit more accurate and no more difficult to use $n - 1$ rather than n.

Plastic Flow—Columns—Sustained Load

For the theory of stress distribution change in reinforced concrete columns under sustained load we are indebted to W. H. Glanville (15) of the Building Research Station of England. He presents

$$f_c = \frac{f_o}{e^{c/b}} \qquad \text{or} \qquad f_c = \frac{f_o}{e^{\left[\frac{cA_sE_sE_c}{A_cE_c+A_sE_s}\right]}} \qquad \text{where}$$

c = the unit plastic flow for a unit stress of unity
f_o = the unit stress in the concrete at time of loading
f_c = the unit stress in the concrete after a time
e + the Naperain base

An expression similar to this is derived in detail in a discussion of reference number 22 by the author, where the formula appears in the form:

$$f_{cx} \qquad \text{or} \qquad f_c \text{ by Glanville} = \frac{f_c \text{ or } f_o \text{ by Glanville}}{e^{\left[\frac{pE_sCx^{1/a}}{p(n-1)+1}\right]}}$$

$Cx^{1/a}$ replaces Glanville's term c.

It is possible to obtain fairly accurate values for f_{cx} by means of what might be termed the modular ratio change procedure. A new modulus of elasticity for concrete may be expressed by including the plastic with the elastic deformation.

$$M_{cx} = \frac{f_{cx}}{\delta_c + Cx^{1/a}}$$

A new modular ratio $\frac{M_s}{M_{cx}} = N$ or $N = n + E_s Cx^{1/a}$ may be used to find the unit stresses

$$f_{cx} = \frac{p(n-1)+1}{p(N-1)+1} f_c$$

The error in the increase of f_{sx} over f_s of this approximation according to Glanville (15) is never more than 23 per cent. This maximum error occurs where $\frac{CpE_s x^{1/a}}{p(n-1)+1}$ equals 1.8 or 2.0.

Columns-Sustained Strain

The situation in this problem is unique in that variations in stress are to be computed on the basis of non-changing deformations. From an elastic standpoint this is absurd. In order to consider this problem infinitesimal deformations and instantaneous retrievings of original form must be imagined. To maintain no change in the deformation the rate of plastic flow must just equal the rate of elastic recovery. This problem is the same for plain concrete and reinforced concrete. The presence of reinforcing steel has no effect since no change in the deformation of a member causes no change in the stress in the steel.

An expression for the unit stress in the concrete after a time has been worked out by C. S. Whitney (8).

$$f_c = \frac{f_o}{e^{(EcC)}} \qquad \text{or} \qquad f_{cx} = \frac{f_c}{e^{(EcCx^{1/a})}}$$

Beams—Sustained Load

When considering the problem of solving for deflections of singly reinforced beams under sustained load, the following conditions are apparent. The concrete and steel are separate and therefore act separately, which is different from the condition of columns; there is no continuing transfer of stress from a plastic to a non-plastic material. Since the assumption has been made that the plastic flow is proportional to the stress, the straight line principal may be carried through

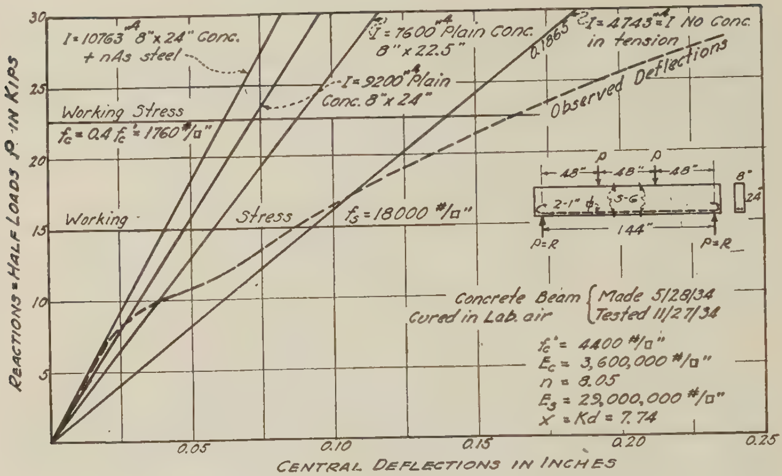

FIG. 4

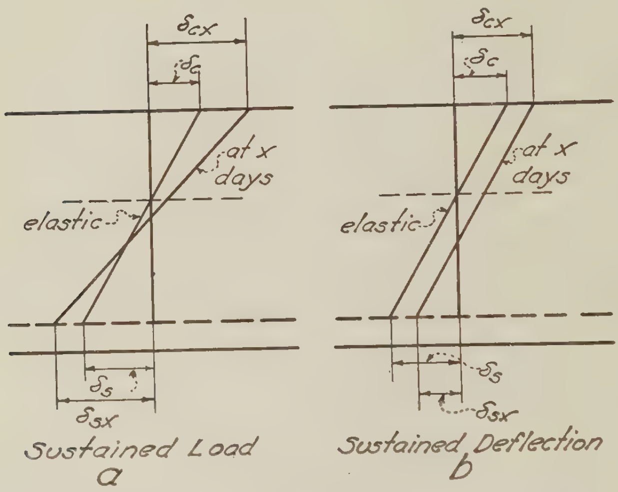

FIG. 5

into plastic conditions with accuracies as great as for the elastic. Fig. 5(a) shows what might be expected to take place. The problem may then be solved in the usual manner as for elastic conditions when

$$N = n + E_s C x^{1/a}$$

is substituted for the usual value, n. The data obtained by this procedure is then available for solving for unit stresses.

As discussed under the head of shrinkage the difficulty with this procedure is the uncertainty, on account of cracks, as to what should be considered to be the working depth of concrete. For this reason it is feasible only to assume some qd depth for computing the location of the neutral axis and the value for the moment of inertia.

This is the theory presented by Oscar Faber (10) and its practical accuracy is illustrated by Fig. 9 of the test description.

Beams—Sustained Strain

The action at a section of a beam under sustained deflection is illustrated in Fig. 5b in contrast to that for sustained load Fig. 5a. In both cases the neutral axis moves toward the tension steel because of the reduction of the effective concrete modulus or the increase of N. The increasing deflection in the case of sustained load causes an increased tipping of the line. The stress in the steel increases because of the reducing of the arm of the internal couple. In the case of the sustained deflection the straight line after a time must remain parallel to the zero time line in order to maintain constant deflection. As the load must reduce, the unit stress in the steel is likely to reduce unless the decrease in the arm of the internal couple more than offsets this load reduction. The parallel movement of the line would sum up for the whole beam as a total shortening of the beam. This change would probably not be large enough to be of any real consequence.

Considering the case of the sustained deflection, it is evident that the moment of the M/EI diagram must remain constant notwithstanding that M, E_c, and I, change with time. The change to E_{cx} as evidenced by the expression for N, depends entirely upon the plastic flow values C and a and not upon the amount of stress. For any loaded beam of uniform cross-section the only change from point to point on the beam is the amount or degree of stress. From this condition it follows that the effective modulus of elasticity of the concrete, E_{cx}, is constant from end to end of the beam for any one time x. The location of the neutral axis and the moment of inertia are therefore also constant. If E_s, E_{cx}, I_c and consequently N are constant from end to end, and the moment of the area of the moment diagram must remain constant, then the shape and size of the M/EI diagram will remain the same.* The shape of the elastic line of the beam must also remain constant, and its slope must be constant at all points. This reasoning shows that the loading at any time of a beam loaded to maintain constant deflection may be computed from N, qd, the general dimensions of the beam and the loading and span conditions.

*See test confirmation page 176.

The procedure then is the same as that for sustained load excepting that M must be solved for in the moment areas expression instead of the deflection which is constant and obtainable from the elastic conditions.

The similarity between the case of the column under sustained strain and the beam under sustained deflection may be so striking as to suggest a question as to the accuracy of this modular ratio change procedure. It seems, however, that the adoption of the straight line principle answers this question. The method appears to be vindicated by the tests on beams reported herewith.

*Plastic Flow Tests on Beams**

Though many tests have been made to determine the rates of plastic flow on plain concrete in direct compression, little study has been made on the rates of plastic flow and change of stress for beams under sustained deformations or deflections.

The object of these tests was to obtain information on the rate of load release for constant deflection on a beam, and to compare this figure with beam deflection variations for sustained load and plastic flow for sustained load on plain concrete.

Four simply reinforced beams, 6 in. x 9 in. x 13 ft., and three prisms, 4 in x 4 in. x 2 ft. 1 in., were made simultaneously of concrete proportioned 1:3.8:3.94 by weight with a cement water ratio by weight of 1.15. The ultimate strength average for four standard concrete compression specimens at 27 days under standard curing was 3080 p.s.i. The modulus of elasticity for the concrete, average of two was 3,120,000 p.s.i. This concrete is listed as No. 26 in Table 1.

One of these beams was used for constant loading and a second for constant deflection. The other two beams were used as reference beams so as to eliminate the effect of dead load and, as much as possible the effects of temperature and humidity changes. The set-up is shown in Fig. 6. The two near beams, O and P, were used for the sustained deflection test and the two far ones, Q and R, for the constant load test. Water in steel drums, A, made up the most of the load for the near beam and two pieces of steel, B, for the far beam. The large steel beam, C, served merely as a support for the reactions. The loading straps went through holes in the web of this steel beam. Ames dials D and E served to measure the deflections. An electrical contact contrivance, F, sensitive to 0.00005 of an inch, with red and green lights, G, aided in maintaining the live or concentrated load de-

*This series of tests was performed by G. E. Large, Associate Professor of civil engineering, The Ohio State University. Associated with Mr. Large in this work were C. T. Morris, Professor of civil engineering, who offered the original suggestion and acted as advisor, and J. R. Shank. H. J. Hoffman, Station draftsman, rendered valuable assistance.

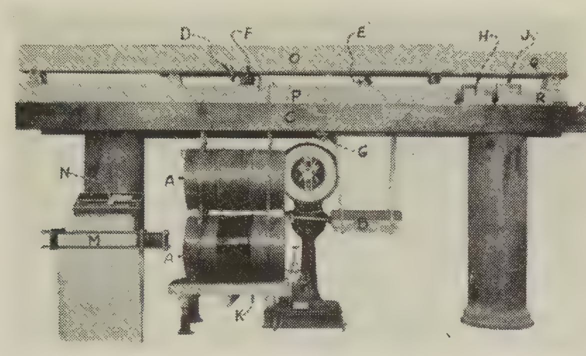

FIG. 6

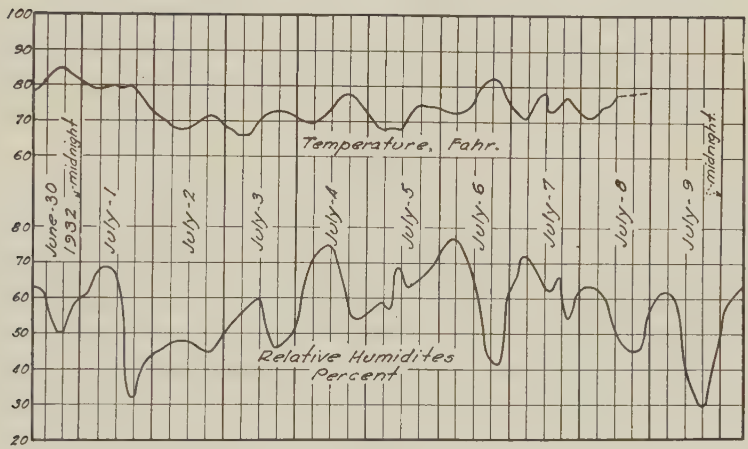

FIG. 7

flection constant. Recording thermometer and hygrometer, *H* and *J*, produced data on the temperature and relative humidity for the time of the test. See Fig. 7.

When the tests were started and the supporting frame *K* under the drums was removed, the deflection of the near beam was maintained constant by letting water out of the drums, *A*. Readings on water release and on the deflection of the far beam were taken continuously day and night for ten days. The loadings on the far beam, concentrated at two points each 12 inches from the mid-span, amounted to 534 pounds each. The span was 12 ft. The loadings at the start for the near beam or constant deflection test were the same and similarly placed.

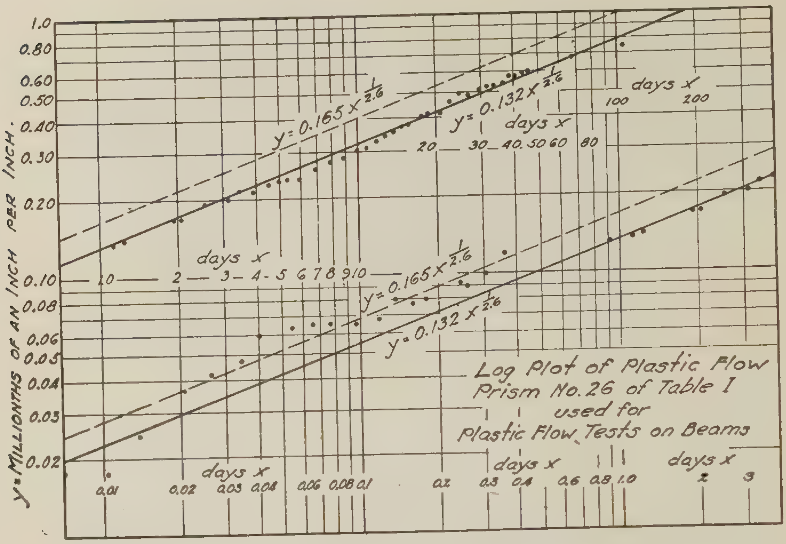

FIG. 8

One of the 4 in. x 4 in. prisms was placed in the spring loading device *M* at the beginning of the test and readings were taken on it by means of the fulcrum plate strain-gage, *N*. Readings under conditions of no load were taken at the same time on one of the other prisms which acted as a control specimen and served to eliminate the effect of temperature and humidity. These readings were continued for a greater time than ten days. The results are shown on the log plat Fig. 8. Of the two curves drawn, the lower one, $y = 0.132\ x^{1/2.6}$, is given under No. 26 of Table 1, and the other one, which will be spoken of later in the discussion, seemed to fit the constant deflection test.

The results of these tests are shown on Fig. 9 and 10. Circles represent observed values and curves are drawn from theoretical calculations as indicated in this paper. The observed elastic or initial deflections served as bases for obtaining the average crack penetrations or the values q. The observed elastic deflections immediately after loading were 0.067 inches for the constant load test and 0.056 inches for the constant deflection test. These two deflections are obtained from computations when the values for q are taken as 1.03 and 1.12 respectively, and their corresponding working depths qd are 8.00 and 8.68 inches. Adjustment of the fraction q was found to be necessary for the constant load test to produce the conformity shown on Fig. 9

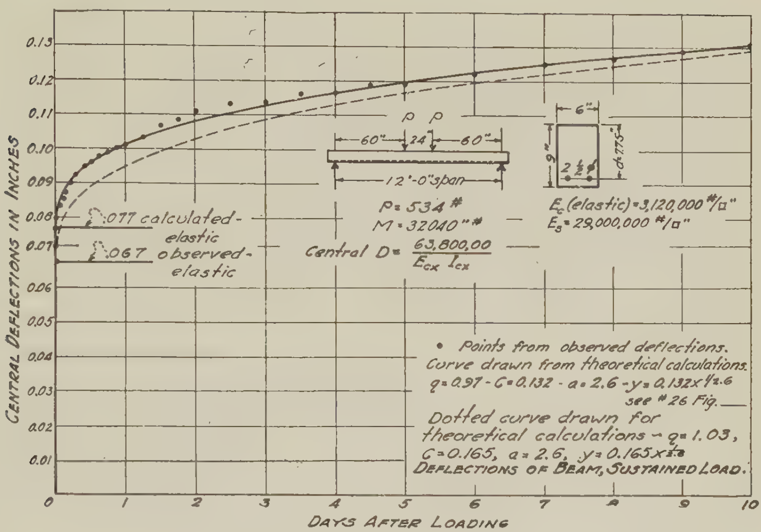

Fig. 9

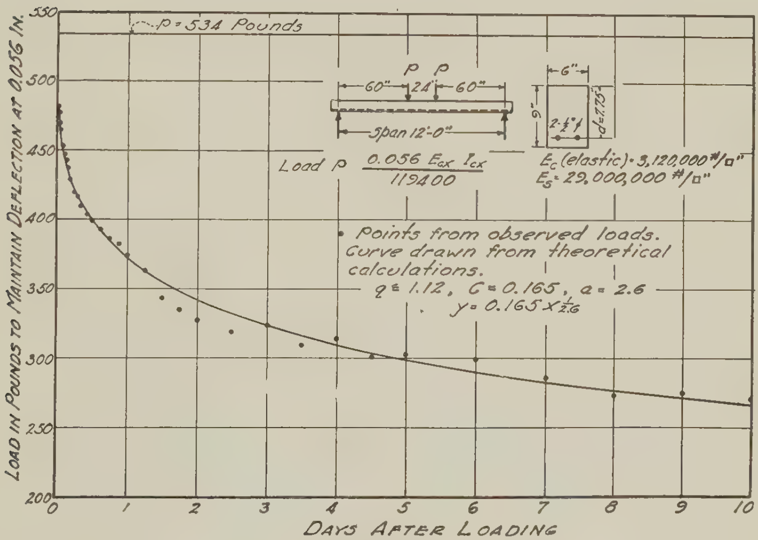

Fig. 10

and of the value C in $y = Cx^{1/a}$ in the constant deflection test, Fig. 10.

A curve computed and drawn for constant load, $P = 534$, $q = 1.03$, $E_c = 3{,}120{,}000$ p.s.i., $E_s = 29{,}000{,}000$ p.s.i. and $y = 0.132x^{1/2.6}$ would

fall below the observed points on Fig. 9. If q is changed to 0.97 the curve as shown is produced, but the elastic deflection would be 0.077 inches. The first three points plotted after the initial at 0.067 inches show that a progressive cracking took place in the first fifteen or twenty minutes and a curve drawn for the points plotted might just as well have started at 0.077. The original data showed that the deflection 0.077 inches occurred 15 minutes or about 0.01 days after initial loading. The dashed curve is drawn to conform to a change of C in $y = Cx^{1/a}$ which was necessary for the theoretical curve for the sustained deflection test.

The results for the sustained deflection test are shown in Fig. 10. A theoretical curve $q = 1.12$, $E_c = 3{,}120{,}000$ p.s.i., $E_s = 29{,}000{,}000$ p.s.i., $C = 0.132$ and $a = 2.6$ falls considerably above these points, and an adjustment considering crack penetration, as was made for the sustained load test was not possible, in this case. To reduce the value of q was in the wrong direction and the full nine inches of depth was not sufficient to bring the theoretical curve down to the points observed. The curve as shown is drawn for $q = 1.12$, $E_c = 3{,}120{,}000$ p.s.i., $E_c = 29{,}000{,}000$ p.s.i., $C = 0.165$ (instead of 0.132) and $a = 2.6$. The formula for plastic flow for plain concrete had to be changed. This formula $y = 0.165x^{1/2.6}$ is shown on the log plot Fig. 9 as the dashed or upper line. It is of interest that this curve suits the observed data fairly well for the first half day. The dashed curve on Fig. 9 is drawn to suit this change in C and without any correction for q.

At the beginning of the test of the constant deflection beam a sensitive clinometer was applied to the ends of the beam to see if there might be any change in the slope of the elastic line as time went on. Toward the end of the first day these measurements were discontinued because no change was apparent. This confirmed the statements of page 171, lines 32-36.

These tests show that the theory given in this paper conforms to test observations even though the problem of obtaining the correct values for the plastic flow formula is not yet completely solved. The uncertainty of obtaining correct information concerning the crack penetration will probably always remain the greatest source of error when trying to estimate deflections and load release. It should be noted that the adjustment of the plastic flow constant C from 0.132 to 0.165 is contrary to the effect of size of specimen as given by Davis, Davis, and Hamilton (14) on Fig. 7, page 368. The 4 in. x 4 in. prism showed the lower plastic flow in spite of its being the smaller cross-section.

Beams Reinforced for Compression

The tests just described and the theoretical discussions for beams already given deal with conditions wherein the steel is generally outside the region of working concrete. The doubly reinforced beam or beam reinforced for compression is not in this class, which brings the theory for columns to mind. The complexity and difficulty involved in developing a theory for doubly reinforced sections in bending, along the lines laid out by Glanville for columns, appear to be very great and we are therefore driven to consider the degree of approximation, when using the method of modular ratio change, used here for singly reinforced beams.

Glanville (15) after developing his theory for columns considers the degree of the approximation when the modular ratio change method is used for columns. He finds that the maximum error in the changes of the unit stresses after the elastic action has taken place is 23 per cent. The amount of this error depends upon a term c/b used by him which is $\dfrac{pE_sCx^{1/a}}{p(n—1)+1}$, $Cx^{1/a}$ corresponding to his value c and $\dfrac{p(n—1)+1}{pE_s}$ to b. The former or numerator is the unit plastic flow for unit unit-stress and the latter is a term obtained from the dimensional and elastic properties of the column. The former varies with time and the latter is constant. This maximum of 23 per cent obtains when c/b is around 1.75 or 2.00. The error which is obviously zero at zero time increases quite rapidly to the value of 1.75. After the value 2.00, the error reduces with advancing value of c/b and approaches zero when c/b approaches infinity.

It appears that the modular ratio change method might be used as a good approximation, and if conservative results are required the changes in the unit stresses might be increased by 20 per cent. In any practical problem, uncertainties about the crack penetration, the real elastic properties of the concrete, and the plastic flow expression are great enough so that it is hardly practical to use any theory more complex than the modular ratio change method.

Bending and Direct Stress Conditions

The problem in elastic calculations for bending and direct stress combinations is complex largely because of the assumption that no concrete acts in tension. This assumption is responsible for the cubic equation. After the line of zero combined fiber stress is found it is necessary only to compute the stresses at any fiber separately for bend-

ing and direct stress and to combine them. The bending stress is best found by the moment of inertia of transformed section method. When considering deflections or stresses arising from deflections the assumption that no concrete acts in tension is untenable and some value for q must be assumed. Here again as for the doubly reinforced beam, changes in unit stresses due to plastic flow may be calculated by the modular ratio change method for the bending and by the more accurate theory of Glanville for the direct stress, and combined, with a degree of accuracy that is apparently the best that may be hoped for. Deflections may be computed on the same basis.

ACKNOWLEDGEMENTS

Credit is due to Clyde T. Morris, Professor of Structural Engineering, who offered the original suggestion for the project and whose comments during the investigation were of the greatest assistance in the work done at The Ohio State University. Credit is also due to G. E. Large, Associate Professor, and H. J. Hoffman for their work on the beam tests. Two graduate students, J. R. Clifton and L. L. Sammet, made valuable preliminary calculations on the type of curve equation to use and on the applications to mechanics. Twenty undergraduate students whose initials appear in the second column of Table 1 assisted in carrying on the various plastic flow tests.

Materials for the test set-up and special aggregate materials were furnished by The Carnegie Steel Company, The Standard Slag Company, and the Hydraulic-Press Brick Company among others.

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For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June 1936. Discussion should reach the Secretary by April 1, 1936.*

Building Regulations For Reinforced Concrete*

Submitted by Committee 501. Standard Building Code

A. W. STEPHENS, CHAIRMAN
AND R. R. ZIPPRODT, SECRETARY

The Tentative Building Regulations for Reinforced Concrete now in use were adopted as a Tentative Standard of the American Concrete Institute at the 24th annual convention, Philadelphia, Febrary 28, 29 and March 1, 1928.

The eight intervening years have seen many advances in design and construction. Committee 501 of the American Concrete Institute, acting with the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute—in accordance with the agreement reached previous to the 24th annual convention—has revised the Tentative Standard, as adopted in 1928, to bring it into agreement with current practice and developments.

Though, in general, the proposed revised Tentative Standard follows the form of that adopted in 1928, many changes in the requirements have been made. Among the more important are the following:

1. The requirement that all concrete exposed to the weather shall have a minimum ultimate 28-day compressive strength of 3000 p.s.i.

2. The revision of the chapter on Concrete Quality and Working Stresses to conform to recent developments in present day practice.

3. The allowable unit stress in wire mesh or other steel reinforcement, not exceeding ¼-in. in diameter, when used in one-way slabs, has been increased from 20,000 to 30,000 p.s.i., based on recent tests at the University of Delaware.†

4. The addition of a set of allowable unit stresses for concrete having a minimum ultimate 28-day compressive strength of 3750 p.s.i.

5. Important changes have been made in the requirements for flexural computations in the case of structural members spanning in one direction, thereby affording the designing engineer the opportunity

*For presentation 32nd Annual Convention, Chicago, Feb. 25-27, 1936.
†See "Concrete Slabs Reinforced with Welded Wire Fabric" by T. D. Mylrea, in this Journal.

of utilizing the principles of continuity which have come into general acceptance since the issuance of the previous Tentative Standard.

6. The complete revision of the requirements for the design of two-way slabs which are supported on four sides.

7. Changes in the design requirements for columns based upon the very extensive series of tests sponsored by the Institute through Committee 105, Reinforced Concrete Column Investigation; tests were made at Lehigh University and the University of Illinois.

8. Changes have also been made in the requirements for the concrete protection for reinforcement; shear and diagonal tension; bond and anchorage; resistance to wind forces; monolithic walls; and the chapter on flat slabs has been rearranged for more convenient use.

This report represents the work of eleven Sub-Committees of the two main cooperating committees. All sub-committee reports have been approved by a majority of the members of each such sub-committee. Certain sections, not otherwise covered, were referred to the Editorial Sub-Committee with power to act.

Committee 501 submits the revised report with the recommendation that it be adopted as a Tentative Standard.

The complete report of Committee 501, as proposed for adoption as a new Tentative Standard of the Institute, is now available for distribution. One copy will be sent without charge to any Institute member whose request for it is received by the Institute Secretary before June 10, 1936. It will be available also to non-members—and additional copies to members—at 50 cents per copy. Committee 501 will make formal presentation of the report at the 32nd Annual Convention, Chicago, February 25-27, 1936, when the proposed new code will be open to discussion. Priority of consideration will be given to such discussion as is submitted in duplicate in advance of the convention to the Institute Secretary. Action on the report, in reference to the proposal to adopt it as a new Tentative Standard, will be in accordance with the By-Laws, Article 5.

Isteg Steel for Concrete Reinforcement*

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INTRODUCTION

Isteg steel reinforcement, extensively used in Europe, consists of two plain round bars, initially straight and parallel, which have been twisted together cold to form a double helix. During the twisting operation, the ends of the two bars are rigidly restrained against longitudinal movement, thereby stretching the bars during the twisting operation. The Isteg steel is thus cold worked in tension and torsion. The physical properties and shape resulting from this cold working operation render the twin bar "Isteg" superior to the original plain round bars for concrete reinforcement.

To verify the physical properties and behavior of Isteg steel in comparison with the conventional types of concrete reinforcement, a comprehensive test was conducted at the Civil Engineering Testing Laboratories of Columbia University, from September, 1934 to March, 1935. The program included comparative tests on Isteg, plain round and deformed round bars, and comprised tension tests, cold bend tests, rectangular beam tests, T-beam tests, and bond tests.

All of the steel reinforcement for these tests was furnished by the Kalman Steel Corporation, a subsidiary of the Bethlehem Steel Corporation. To insure uniformity in the quality of the steel, all bars were rolled from billets taken from one heat of steel. The physical properties of all the straight bar stock were in accordance with the standard specifications for structural grade billet steel concrete reinforcement bars, A. S. T. M. serial designation A 15-30. The deformed bars were the Bethlehem type.

The Isteg bars were twisted together, at room temperature, to the specified pitch of 12.5 times the diameter of a single bar. It was desired that lengths of 48 ft. be twisted, to minimize the relative effect of longitudinal yield of the machine, but the only machine available was the one used for twisting square bars in which the twisted length was limited to 17 ft. 6 in. The Isteg bars furnished were pro-

*Received by the Institute Secretary, June 17, 1935.
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vided with straight untwisted ends 24 in. long for comparative tension tests on the original material.

The concrete for all tests was made of uniform materials, proportions, curing, and strength, the 69 control cylinders closely checking the uniform strength of approximately 3300 p.s.i. in 28 days.

1. TENSION TESTS

In addition to the determination of the usual physical properties, the tension tests included strain measurements from which the modulus of elasticity and the stress-strain curves were obtained. Both "continuous" load tests (with continuous application of increasing load) and "repeated" load tests (with release of loading at intervals to obtain permanent "set" measurements) were included.

The "useful" limit or "yield strength" of reinforcing steel is defined as the stress producing a permanent elongation of 0.2 per cent or a total elongation under load of 0.4 per cent. These two criteria yield almost identical values. Below this stress, the effect of any plastic yielding upon the usefulness of the material is negligible; beyond this stress, the capacity of the concrete to follow the strain of the steel is impaired. For all bars tested, the "yield strength" was determined from the plotted stress-strain curves. In the case of the hot-rolled rods (plain and deformed bars), this yield strength is not appreciably different from the yield point as determined by the "drop of the beam" method. In the case of the cold worked material (Isteg bars), in which the stress-strain graph is a smooth curve in the region of yield, there is no definite "yield point" and the "drop of the beam" method cannot be used.

The results of the tension tests on the Isteg bars are recorded in Table 1.

TABLE 1—TENSION TESTS ON ISTEG BARS

Isteg Bar Size	Type of Test	Yield Strength p.s.i.	Ultimate Strength p.s.i.
⅜" $\phi\phi$	Repeated	56,500	65,800
⅜" $\phi\phi$	Continuous	56,400	66,900
⅜" $\phi\phi$	Continuous	56,100	66,200
½" $\phi\phi$	Continuous	56,100	65,700
½" $\phi\phi$	Repeated	55,600	65,600
½" $\phi\phi$	Continuous	55,500	66,700
⅝" $\phi\phi$	Repeated	51,700	61,900
⅝" $\phi\phi$	Continuous	51,300	62,400
⅝" $\phi\phi$	Continuous	51,200	61,900
¾" $\phi\phi$	Repeated	50,900	58,150
¾" $\phi\phi$	Continuous	50,800	58,200
¾" $\phi\phi$	Continuous	49,700	58,900

The average value of the elastic modulus E was 30,124,000 for the plain round rods and 22,854,000 for the Isteg bars. Accordingly the value of n for Isteg bars should be taken at ¾ of the value used for the plain rods.

The increase in strength resulting from the Isteg cold-twisting operation is shown in Table 2 of comparative tension tests on plain round bars and Isteg bars.

TABLE 2—RELATIVE STRENGTH OF PLAIN AND TWISTED BARS

(Average Values in Pounds per Square Inch)

Bar Size	Plain Round Bar		Isteg Bar		Per Cent Increase	
	Yield Str.	Ult. Str.	Yield Str.	Ult. Str.	Yield Str.	Ult. Str.
⅜" ϕ	39,900	57,900	56,330	66,300	41.0	14.5
½" ϕ	36,700	56,700	55,730	66,000	51.9	16.1
⅝" ϕ	34,800	53,900	51,400	62,070	47.6	15.1
¾" ϕ	34,000	53,400	50,470	58,420	48.2	9.2
				Average:	47.2	13.7

The comparison (Table 2) shows that, as a result of the cold-working (twisting and stretching), Isteg bars are superior to plain round bars (of the same steel and the same diameter) by 47.2 per cent in yield strength and by 13.7 per cent in ultimate strength. There is a further strength advantage resulting from the usual substitution of smaller diameter Isteg bars for any specified diameter of plain round rods. Thus ⅜ in. $\phi\phi$ Isteg would replace ⅝ in. ϕ plain rounds, yielding (from Table 2) an increase of 62 per cent in yield strength and 23 per cent in ultimate strength; and, similarly, ½ in. $\phi\phi$ Isteg would replace ¾ in. ϕ plain rounds, yielding an increase of 64 per cent in yield strength and 23 per cent in ultimate strength. Since yield strength is the significant tensile property of reinforcing steel, the foregoing comparisons indicate for Isteg reinforcement approximately 63 per cent more efficiency than for ordinary plain round rods.

The tests also indicated that deformed round rods have practically the same tensile properties as plain rounds, with perhaps a 2 per cent increase in yield strength and ultimate strength. With this correction the Isteg reinforcement appears approximately 60 per cent more efficient than deformed round bars.

2. COLD BEND TESTS

Cold bend tests were made in accordance with the standard methods upon thirty-inch lengths of each of the four sizes of Isteg bars, (⅜ in., ½ in., ⅝ in., and ¾ in.) The bending operation was performed in a cold bending machine in which the bar is bent through 180 degrees

around a mandrel. When both elements of the Isteg bar were placed in contact with the mandrel, the mandrel diameter used was equal to the diameter of a single bar element. When the Isteg bar was placed so that only one element was in contact with the mandrel, the mandrel diameter used was equal to the overall bar diameter, i. e., twice the diameter of a single bar element. All of the Isteg specimens were bent cold without indication of cracking.

In preparing the reinforcement for the beams subsequently tested, the Isteg bars were provided at the ends with bent hooks in the customary manner. Nó difficulty was experienced in making these hooked ends, and no appreciable displacement of the elements or distortion of the bars resulted.

3. RECTANGULAR BEAM TESTS

All of the reinforced concrete beams included in this series of tests were 12 in. by 12 in. in cross-section and 9 ft. in span. They were loaded at the one-third points with equal loads in a 200,000 lb. Riehle Universal Testing Machine. Stresses in the concrete and in the steel were measured with strain gages.

One group of beams, designed for approximately equal design loads, had the percentage of reinforcement ranging from 0.45 to 0.62 per cent, depending upon the type of reinforcing bars used. A second group of beams, designed for a higher loading, had the percentage of reinforcement ranging from 0.80 to 1.25 per cent for the respective types of reinforcing bars.

The beams with plain round and deformed bars were designed for a steel stress of 18,000 p.s.i. and $n = 10$. The beams with Isteg reinforcement were designed for a steel stress of 27,000 p.s.i. and $n = 7.5$.

The design stress calculated for the concrete at the design load in the beams of Group 1 was 700 p.s.i. for the beams with plain round and deformed rods and 1015 p.s.i. for the beams with Isteg reinforcement, or 45 per cent higher design stress for the concrete with Isteg steel.

In the beams of Group 2, the concrete design stress at the design load was 1120 p.s.i. for the beams with deformed rods and 1450 p.s.i. for the beams with Isteg reinforcement, or 30 per cent higher design stress for the concrete with Isteg steel.

The comparative results of these beam tests are recorded in Table 3.

In the tests of Group 1 the beams with Isteg bars had 28 per cent less reinforcing steel than the beams with plain round rods but proved 24 per cent stronger in the ultimate load they carried, indicating for Isteg reinforcement a 73 per cent greater efficiency.

TABLE 3—RESULTS OF RECTANGULAR BEAM TESTS

Reinforcement	Per Cent	Max. Load	Max. Observed Steel Stress	Factor of Safety	
				Based on 18000 p.s.i.	Based on 18000/ϕ 27000/$\phi\phi$
Group 1					
⅝" ϕ Plain	.624	22,700	34,800	2.38	2.38
⅝" ϕ Plain	.643	22,600	34,450	2.40	2.40
⅝" ϕ Defd.	.620	25,135	35,750	2.62	2.62
⅝" ϕ Defd.	.623	24,970	35,500	2.56	2.56
⅜" $\phi\phi$ Isteg	.451	28,150	59,500	4.00	2.66
⅜" $\phi\phi$ Isteg	.456	27,950	59,750	3.99	2.66
Group 2					
⅞" ϕ Defd.	1.245	42,700	35,800	2.35	2.35
⅞" ϕ Defd.	1.230	42,000	36,000	2.27	2.27
½" $\phi\phi$ Isteg	.798	45,250	58,400	3.62	2.41
½" $\phi\phi$ Isteg	.820	44,950	59,000	3.61	2.40

Averaging the ratios yielded by the tests of Groups 1 and 2, the beams with Isteg bars had 31 per cent less reinforcing steel than the beams with deformed rods but proved 9 per cent stronger in the ultimate load they carried, a 58 per cent greater efficiency for Isteg steel.

The maximum observed steel stress before failure in the beams with the respective types of reinforcement averaged as follows:

Plain round bars.........34,625 p.s.i.
Deformed bars..........35,762 p.s.i.
Isteg bars..............59,162 p.s.i.

—indicating for Isteg reinforcement in concrete beams a 71 per cent greater efficiency than for plain round bars and 65 per cent over deformed bars.

The observed factors of safety of the beams with the respective types of reinforcement, representing the ratios of ultimate load to design load at a steel stress of 18,000 p.s.i., averaged as follows:

Plain round bars.............. 2.39
Deformed bars................ 2.45
Isteg bars.................... 3.81

On this basis of comparison, Isteg reinforcement in concrete beams yields 59 per cent higher factor of safety than plain round bars and 55 per cent higher factor of safety than deformed bars.

With actual design load based on a design stress of 18,000 p.s.i. for the plain and deformed rods and 27,000 p.s.i. for the Isteg bars (and with 30 to 45 per cent higher design stress for the concrete and n = 7.5 instead of n = 10), the observed factors of safety of the beams with the respective types of reinforcement averaged as follows:

Plain round bars.............. 2.39
Deformed bars................ 2.45
Isteg bars.................... 2.53

These concrete beams reinforced with Isteg bars, designed for a steel stress of 27,000 p.s.i., n = 7.5, and a concrete stress 30 to 45

per cent higher than normally specified, appear superior to the test beams reinforced with plain or deformed bars designed for a steel stress of 18,000 p.s.i., $n = 10$, and the concrete stress as normally specified.

The tests also showed that the ultimate strength of beams is higher than the loads developing the full yield strength of the steel, by the following percentages for the different types of reinforcement:

Plain rounds:	0 to 6%.	Average 3%
Deformed bars:	1 to 11%.	Average 5%
Isteg bars:	9 to 14%.	Average 12%

From these comparative results it appears to be on the safe side to base design stress for the respective types of reinforcement on the relative values of the "yield strength" as above defined. They also show that in comparison with the other types tested, the beams with Isteg reinforcement had a higher margin of ultimate strength above the yield strength of the reinforcement.

This superior margin of ultimate strength of these beams above yield strength of the reinforcement explains why Isteg reinforcement which, in tests already recorded, was found to be 60 to 63 per cent superior to deformed and plain bars in direct tension tests, proves to be 65 to 71 per cent superior to deformed and plain bars in beam tests.

The maximum loads carried by the beams per unit A_s (sectional area of reinforcing steel) yielded the following average ratios:

Plain round bars	1.000
Deformed bars	1.105
Isteg bars	1.760

Accordingly, from the viewpoint of ultimate beam strength per pound of reinforcement required, Isteg steel was here 76 per cent more efficient than plain round rods and 59 per cent more efficient than deformed bars.

In the beams of Group 2 (having high percentages of reinforcement), the maximum observed and calculated stresses in the concrete at the ultimate loads were 3300 observed and 2640 computed in the beams with deformed rods, and 3290 observed and 3510 computed in the beams with Isteg bars. With concrete of equal strength (approximately 3300), the beam with deformed rods failed at a calculated concrete stress of 2640 p.s.i. whereas the beam with Isteg reinforcement (and 34 per cent less reinforcement) did not fail until the calculated concrete stress reached the high value of 3510 p.s.i. This comparison indicates a superior resisting power in the Isteg reinforcement in taking up and sustaining excess beam load after the concrete would normally be expected to fail. It also indicates that, with the

Isteg reinforcement, the actual ultimate capacity of the beams tested was greater than its calculated ultimate strength, and that a beam reinforced with Isteg bars will not fail when the ultimate strength of the concrete is theoretically reached, but will continue to carry increasing load even after the calculated stress in the concrete is 6 per cent above the actual ultimate strength of the concrete and the calculated steel stress is 12 per cent above the actual yield strength of the steel.

The foregoing comparisons of beam tests yield the following respective values for the ratios of superior efficiency of the Isteg reinforcement over the plain and the deformed round bars:

	Superior Efficiency of Isteg	
	over Plain Bars	over Deformed Bars
Ratio of Ultimate Loads divided by Ratio of Steel Used	173%	158%
Ratio of Maximum Observed Steel Stress	171%	165%
Ratio of Ultimate Loads per Unit Section of Steel	176%	159%
Average	173%	161%

Hence, judged in these test comparisons, Isteg steel as beam reinforcement was 73 per cent more efficient than the plain bars and 61 per cent more efficient than the deformed bars.

4. TESTS ON T-BEAMS

The T-beams in this series of tests were 19½ in. wide by 11 in. deep, with 4-in. top flange and 7½-in. stem, and 9 ft. in span. The comparative results of these T-beam tests are recorded in Table 4.

TABLE 4—TESTS ON T-BEAMS

Reinforcement	Per Cent	Max. Load	Max. Observed Steel Stress	Factor of Safety	
				Based on 18000 p.s.i.	Based on 18000/ϕ 27000/$\phi\phi$
1¼″ ϕ Defd.	1.161	52,180	36,700	2.20	2.20
1¼″ ϕ Defd.	1.142	55,000	36,900	2.29	2.29
¾″ $\phi\phi$ Isteg	.843	55,000	53,450	3.28	2.18
¾″ $\phi\phi$ Isteg	.812	57,600	53,400	3.30	2.21

The T-beams with Isteg bars averaged 28 per cent less reinforcing steel than the beams with deformed round rods but averaged 5 per cent stronger in the ultimate load they carried, indicating for the Isteg reinforcement 46 per cent greater efficiency than deformed bars in these T-beams with relatively high percentage of reinforcement.

The maximum observed steel stress before failure in theT-beams with the respective types of reinforcement averaged as follows:

Deformed round bars. . . .36,800 p.s.i.
Isteg bars. . . . . .53,425 p.s.i.

This comparison indicates that Isteg reinforcement in these T-beams (with relatively high percentage of reinforcement) was 45 per cent more efficient than the deformed bars.

The observed factors of safety of the T-beams with the respective types of reinforcement, representing the ratios of ultimate load to design load at a steel stress of 18,000 p.s.i., averaged 2.25 with the deformed bars and 3.29 with the Isteg bars, a 46 per cent superiority for the Isteg bars.

The calculated design stress in the concrete at the actual design load was 1075 p.s.i. in the T-beams with deformed bars and 1430 p.s.i. in the T-beams with Isteg bars, or a 33 per cent higher design stress for the concrete allowed in the design of the T-beams with Isteg reinforcement.

With the actual design load for the T-beams based on a design stress of 18,000 p.s.i. for the deformed bars and 27,000 p.s.i. for the Isteg bars (with 33 per cent higher design stress for the concrete and $n = 7.5$ instead of $n = 10$), the observed factors of safety of the T-beams were found to be practically identical with the two types of reinforcement.

From this test comparison it appears that concrete T-beams reinforced with Isteg bars, designed for a steel stress of 27,000 p.s.i., $n = 7.5$, and a concrete stress 33 per cent higher than normally specified are as safe as T-beams reinforced with deformed bars designed for a steel stress of 18,000 p.s.i. $n = 10$, and the concrete stress as normally specified.

The tests also indicated that the ultimate strength of T-beams is higher than the loads developing the full "yield strength" of the steel by 2 per cent in the T-beams with deformed bars and by 9 per cent in the T-beams with Isteg reinforcement. This comparison confirms the justification and safety of basing design stress for respective types of reinforcement on the relative values of the "yield strength." It also confirms the conclusion that, in comparison with other types, beams with Isteg reinforcement have a higher margin of ultimate strength above the "yield strength" of the reinforcement.

The maximum loads carried by the T-beams (of these tests) per unit A_s (sectional area of reinforcing steel) are represented by ratios of 1.00 and 1.48 for the T-beams with deformed and Isteg bars, respectively. Accordingly, from the viewpoint of ultimate beam strength per pound of reinforcement required, Isteg steel (in T-beams) was 48 per cent more efficient than the deformed bars.

Despite the fact that the calculated maximum concrete stress at ultimate load was 2425 p.s.i. for the T-beams with deformed bars and 3245 (or 34 per cent higher) for the T-beams with Isteg bars, the actual observed maximum concrete stress was almost identical in all of the T-beams (3288 with the deformed bars and 3285 with the Isteg bars). The departure in ultimate stress behavior of reinforced concrete beams from theoretical assumptions favors the beams with Isteg reinforcement and gives them a greater relative strength than is indicated by theory.

5. CONDITION OF BEAMS AT FAILURE

The cracks that developed in the beams (with each type of reinforcement) under working loads were all too fine to show in photographs made.

The beams with plain and deformed bars developed a few major cracks near the center which, at failure, opened up from 0.20 to 0.30 in. due to local bond failure and yielding of the steel.

The beams with Isteg steel developed finer cracks which were more uniformly distributed over the length of the beam and which gradually opened up to about 0.10 to 0.15 in. in the middle third of the span just before ultimate failure.

6. BOND TESTS

The bond or "pull-out" tests were arranged in four groups, each containing approximately "equivalent" sizes of the various types of reinforcing bars in order to compare the sizes that would normally be used in mutual substitution.

All bars were 24 in. long and were embedded 8 in. in concrete blocks of 8-in. diameter. This length of anchorage is inadequate to do justice to the Isteg bars of the larger sizes (those in Groups 2, 3, and 4), inasmuch as a length equal to 1.6 times the pitch (or approximately 20 diameters) should be embedded properly to show the full advantage in slip resistance afforded by the helical form. With insufficient length of anchorage, some unscrewing of the test bar is permitted by the untwisting of the free length outside of the concrete, and such untwisting was actually observed at the higher loads. In actual use in beams, the full length of bar is embedded in concrete and there is no free length to permit untwisting.

The results of the bond tests are compared in Table 5.

The test comparisons in Table 5 show that, despite the smaller sections of the equivalent Isteg bars (figured at the higher stress of 27,000 p.s.i. to replace plain and deformed rods at 18,000 p.s.i.) the Isteg bars showed 43 per cent higher bond resistance than plain round bars and 23 per cent higher bond resistance than deformed bars.

TABLE 5—BOND TESTS

Test Bar	Load at First Slip		Stress at First Slip	
	p.s.i.	Ratio	p.s.i.	Ratio
Group 1—⅜″ φφ Isteg	1164	1.70	42,000	2.37
⅝″ φ Plain	685	1.00	17,760	1.00
⅝″ φ Defd.	879	1.28	23,200	1.31
Group 2—½″ φφ Isteg	1406	1.27	28,920	1.45
¾″ φ Plain	1110	1.00	20,000	1.00
⅞″ φ Defd.	1120	1.01	14,880	0.74
Group 3—⅝″ φφ Isteg	1460	1.22	19,040	1.57
1″ φ Plain	1192	1.00	12,160	1.00
1″ φ Defd.	1407	1.18	14,320	1.18
Group 4—¾″ φφ Isteg	2105	1.53	18,960	2.02
1¼″ φ Defd.	1600	1.16	10,160	1.08
Average— (4 Groups) Isteg		1.43		1.85
Plain		1.00		1.00
Defd.		1.16		1.08

If the results of Groups 2, 3 and 4 are omitted, on account of insufficient anchorage length (with excessive free length) to show the full advantages of Isteg bars, the tests show that the Isteg bars had 70 per cent higher bond resistance than the plain round bars and 33 per cent higher bond resistance than the deformed bars.

In the foregoing comparisons it is important to remember that the Isteg bars developing these higher values of total bond resistance are approximately 33 per cent smaller in cross-section than the bars with which they are being compared. The Isteg bars show these superior values of total bond resistance over and above their handicap by the fact that the other bars are 50 per cent larger in cross-section.

Table 5 also records the relative bond resistance in terms of the stress in the bars at first slip. This comparison shows that the Isteg bars were 85 per cent more efficient in bond resistance than the plain round bars and 71 per cent more efficient in bond resistance than the deformed bars.

If, for the reasons previously outlined, the results of Groups 2, 3, and 4 are omitted from this comparison, the tabulated stress ratios show that the Isteg bars were 137 per cent more efficient in bond resistance than the plain round bars and 81 per cent more efficient in bond resistance than the deformed bars.

It appears from these tests that Isteg bars, even after reducing their size for a unit stress of 27,000 p.s.i. as against 18,000 p.s.i. for plain and deformed round bars and 20,000 p.s.i. for twisted square bars, will yield a higher total value of bond resistance than any other type of reinforcement in use.

7. INTERMEDIATE GRADE STEEL

A number of Isteg bars were made of intermediate grade steel—⅜ in, ½ in., ⅝ in. and ¾ in. diameter. These were tested in tension

to obtain a comparison of Isteg bars and plain round bars of this material.

The yield strength of the original round bars ranged from 41,250 to 48,400 p.s.i. The yield strength of the corresponding Isteg bars ranged from 59,500 to 65,000 p.s.i. The average increase in yield strength was 40 per cent.

The ultimate strength of the original round bars ranged from 73,300 to 77,800 p.s.i. The ultimate strength of the corresponding Isteg bars ranged from 70,400 to 81,200 p.s.i. The average increase in ultimate strength was 2.4 per cent.

These tests showed that Isteg bars can also be made advantageously of intermediate grade steel and the yield strength (which is the significant characteristic of reinforcing steel) thereby be increased 40 per cent.

There appears to be a further strength advantage resulting from the usual substitution of smaller diameter Isteg bars for any specified diameter of plain or deformed round rods. If bars of equivalent sizes are compared instead of bars of equal diameters, the tests on the bars made of intermediate grade steel show that the Isteg bars are superior to the equivalent plain round bars of the same material by approximately 54 per cent in yield strength and 10 per cent in ultimate strength (in p.s.i.). Since yield strength is the significant tensile property of reinforcing steel, the foregoing comparison showed the Isteg reinforcement made of intermediate grade steel approximately 54 per cent more efficient than the plain round bars made of intermediate grade steel.

8. CONCRETE DESIGN STRESS

The concrete beam tests showed that the beams reinforced with Isteg steel, designed with a 50 per cent increase in steel stress, $n = 7.5$, and an increase of 30 per cent or more in concrete stress were safer than beams reinforced with plain or deformed bars designed for $n = 10$ and the steel and concrete stresses normally specified. Under these conditions, the rectangular beam tests showed a permissible increase of 30 to 45 per cent in concrete design stress when Isteg reinforcement was used, and the T-beam tests showed a corresponding permissible increase of 33 per cent in concrete design stress.

Calculations based on the test results show that, if the design value of n is not reduced for the beams with Isteg reinforcement, the concrete design stress may be increased 17 per cent and the steel stress 50 per cent to obtain beams of equal or greater strength than beams

reinforced with plain or deformed bars designed with the steel and concrete stresses normally specified.

Similar results and conclusions of tests made in other countries have been the basis of governmental decrees approving an increase of 15 per cent in concrete design stress together with an increase of 50 per cent in steel design stress when Isteg reinforcement is used.

CONCLUSIONS

1. Isteg bars made from the usual structural grade billet steel concrete reinforcement rods are 60 to 63 per cent higher in yield strength.

2. Isteg bars give no difficulty in cold bending.

3. Isteg steel as beam reinforcement is 73 per cent more efficient than plain bars and 61 per cent more efficient than deformed bars.

4. Beams reinforced with Isteg bars possess an extra reserve of resistance and capacity at ultimate loads.

5. Beams designed with Isteg steel at 27,000 p.s.i. and with a 15 per cent increase in concrete stress (without reducing the assumed value of n) will have a higher factor of safety than beams designed with plain or deformed bars at 18,000 p.s.i. and with the usually specified concrete stresses.

6. Isteg bars have a bond resistance 71 to 137 per cent higher than plain and deformed bars and, after reducing the Isteg bars by 33 per cent in section for the recommended higher unit stress, the net superiority in bond resistance is still 23 to 70 percent.

7. Isteg bars can also advantageously be made from intermediate grade steel, yielding a 54 per cent increase in efficiency over plain bars of that material.

The foregoing conclusions from the tests made at Columbia University are confirmed by authoritative tests made at government, university, and private testing laboratories in European countries, including England, France, Germany, Austria, and Switzerland.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June 1936. Discussion should reach the Secretary by April 1, 1936.*

Load Performance Tests of Precast Joist-Precast Slab Floor Construction*

BY R. E. COPELAND†

INTRODUCTION

The tests reported herein were conducted on a type of light weight concrete floor constructed of precast reinforced concrete slab sections laid on and united to precast reinforced concrete joists. Figure 1 shows one of several construction methods.

The precast joist-precast slab floor has been used in a number of structures including a considerable group of houses recently erected for the Tennessee Valley Authority. It represents an alternate application of the precast concrete floor joist which also is used in conjunction with the cast-in-place slab. The structural performance of the latter combination has been previously reported in the Journal.[1]

GENERAL DESIGN CONSIDERATIONS

The design of an economical precast floor system imposes a number of considerations for study. While no definite design load limit was set, it was assumed that such floors are better adapted to use in residences, apartments and other light load structures than to large structures or heavy loadings. The primary objective then is to produce for this class of buildings a fire resistant and structurally adequate floor construction with the least volume, weight and cost of material. It also is essential that the precast members be adaptable to efficient methods of manufacture and erection.

Experience and study indicated interesting possibilities of meeting these basic requirements with the combination of precast joists and precast slabs. It was further indicated that with proper bonding details, the slab and joist would act integrally or in the manner of a T-section. This was considered a desirable feature because it effects a more efficient design and saves in the cost and weight of materials. It appeared that T-beam action was largely a matter of building a

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[1]"Some Tests of Load Capacity of Floors Made with Precast Concrete Joists," by R. E. Copeland and P. M. Woodworth, Journal Amer. Concrete Inst., March-April 1934, *Proceedings*, Vol. 30, p. 311.

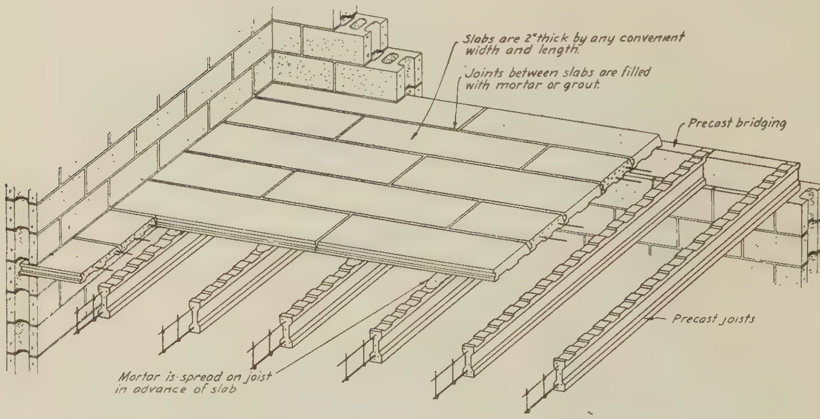


FIG. 1—PRINCIPAL CONSTRUCTION DETAILS OF PRECAST JOIST—PRECAST SLAB FLOOR

strong and rigid joint between the slab and joist sections and also of producing a continuous slab by thoroughly grouting the joints between slab units.

The bond joint between slab and joists is stressed principally by shear which will vary in unit intensity from about 30 to 100 p.s.i. for the usual design loads and spans in light occupancy structures. Occasionally this range will be exceeded. Obviously, the ultimate shear resistance of the bond joint should provide an adequate safety factor and should be capable of developing the yield point strength of the joist reinforcement.

The problem, therefore, seemed to be primarily one of bond joint design and several different types were selected for study in this test program.

OUTLINE OF TESTS

Information regarding the scope and general details of the test program is given in Table 1. Seventeen large panels were tested. The variables included type of aggregate, composition and strength of the bond mortar, bond joint design, texture of the mortar contact surfaces at the bond joint, span of panel and type of loading. Haydite and sand and gravel were selected as being representative of the light weight and standard weight aggregates. The two types of mortars used for the bond joints are commonly referred to as cement mortar and cement-lime mortar. The bond joint designs included those de-

TABLE 1—OUTLINE OF TEST PANELS

All panels were 4 ft. wide by 14 ft. clear span except No. 16 which was of 20 ft. clear span.
8-in. joists spaced 24 in. o. c. used for all panels except No. 16, for which 10-in. joists were used.
Cement mortar consisted of 1 vol. portland cement, 0.10 vol. hydrated lime, 3 vol. sand. Cement-lime mortar consisted of 1 vol. portland cement, 1 vol. hydrated lime, 6 vol. sand.
All panels tested with uniformly distributed loading excepting No. 17 where load was applied at middle and quarter points of span.

Panel No.	Type of Aggregate	Type of Bond Joint	Type of Mortar	Texture of Mortar Contact Surfaces	
				Joist	Slab
1	Haydite	1	Cement	Rough	Smooth
2	Sand and Gravel	1	Cement	Rough	Smooth
3	Sand and Gravel	1	Cement-lime	Rough	Smooth
4	Sand and Gravel	1	Cement	Rough	Rough
5	Haydite	1	Cement	Rough	Rough
6	Sand and Gravel	1	Cement-lime	Rough	Rough
7	Haydite	2	Cement	Smooth	Smooth
8	Sand and Gravel	2	Cement	Smooth	Smooth
9	Sand and Gravel	2	Cement-lime	Smooth	Smooth
10	Haydite	3	Cement	Smooth	Smooth
11	Sand and Gravel	4	Cement	Smooth	Smooth
12	Sand and Gravel	5	Cement	Smooth	Smooth
13	Sand and Gravel	5	Cement-lime	Smooth	Smooth
14	Sand and Gravel	6	Cement	Rough	Smooth
15	Sand and Gravel	6	Cement-lime	Rough	Smooth
16	Sand and Gravel	2	Cement	Smooth	Smooth
17	Sand and Gravel	5	Cement	Smooth	Smooth

TABLE 2—DETAILS OF CONCRETE MIXTURES AND COMPRESSIVE STRENGTHS

Compressive tests on 3 x 6-in. cylinders in air dry condition at time of test of specimens. Cylinders cured in same manner as specimens; 7 days damp, remainder in air.

Type of Aggregate	Quantities by Dry Rodded Volume, Cu. Ft. Per Sack of Cement				Unit Wt. Combined Aggregate Lb. Per Cu. Ft.	Fineness Modulus Combined Aggregate	Average Comp. Strength p.s.i.	Used For
	Separated Aggregate			Combined Aggregate				
	Sand 0-4	Fine 0-4	Coarse 4-⅜					
Haydite	.33	2.03	.77	2.9	63	4.05	5050	Joists and slabs of panels 1, 5, 7, 10
Sand and Gravel	2.00		1.00	2.6	124	3.73	6130	Joists of panels 2, 3, 4, 6, 8, 9, 11, 12, 13, 14, 15, 16, 17
Sand and Gravel	1.60		1.80	2.9	125	4.38	6660	Slabs of panels 2, 3, 4, 6, 8, 9, 11, 12, 13, 14, 15, 16, 17

pending on cement bond alone and types providing mechanical bond in addition to cement bond. The mortar contact surfaces were made rough for some panels and smooth for others. All panels were of 14 ft. clear span except one of 20 ft. span. All panels were tested with uniformly distributed load except one panel where the load was applied at the mid-span and quarter points.

DESCRIPTION OF TEST PANELS

Concrete Mixtures—The details of the concrete mixtures and compressive strengths of the control cylinders are given in Table 2. Higher strengths were obtained than in the series of precast joist tests previously reported which is somewhat inadequately ascribed to slight changes in the mix proportions, particularly the use of a lower water-cement ratio together with a more adequate damp curing period. The cement was a mixture of equal parts of four brands of standard portland cement purchased from local dealers. Concrete was hand-spaded and compacted into the molds.

Mortar Mixtures—The mortar materials were proportioned by loose volume. The cement mortar consisted of one volume cement, 0.10 volume hydrated lime and 3 volumes sand, and had a compressive strength of 2800 p.s.i. The cement-lime mortar consisted of one volume cement, one volume hydrated lime and 6 volumes sand, and had a compressive strength of 1340 p.s.i. For all panels excepting No. 14 and No. 15, the mortar used in the joints between slabs consisted of a 1:1 mixture of cement and sand. This mortar had a compressive strength of 4230 p.s.i. Slab joints in panels 14 and 15 were made of the same mortar as used for bond joints in these panels.

The mortar strengths are based on tests of air dry 3 x 6-in. cylinders at the time of the respective panel test.

Design and Construction of Panels—Typical details of the test panels are shown in Fig. 2. Details of the joist reinforcement are shown in Fig. 3, and the different types of slab sections and bond joints are shown in Fig. 4 and 5.

Joist reinforcement design was based on the usual flexural formulas and the following working stresses: f_s = 20,000 p.s.i.; f_v = 16,000 p.s.i.; v_c = 60 p.s.i.; f_c = 1,200 p.s.i.

Live load of 40 lb. per sq. ft. and dead load of 47 lb. per sq. ft. were assumed. The latter includes an allowance for floor finish, ceiling plaster and partitions.

Stresses due to the dead weight of the joists and slabs were based on the independent joist section. Stresses due to live load and the allowed extra dead load were based on the T-section. The computed required area of tensile reinforcement is 0.37 sq. in. for the 14 ft. panels and 0.61 sq. in. for the 20 ft. panel. The used areas were respectively 0.44 sq. in. and 0.60 sq. in.

The slab sections were of two sizes. All panels except No. 14 and No. 15 were built of slabs 12 in. wide, 48 in. long and 2 in. thick, reinforced by two ¼ in. bars placed about ½ in. from the bottom face. This "plank" type slab may be made in different widths and lengths.

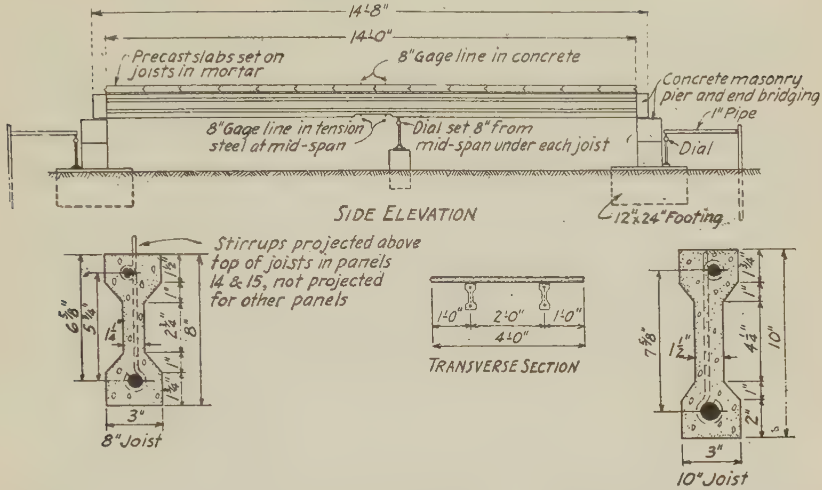


FIG. 2—PRINCIPAL DETAILS OF TYPICAL PANEL AND JOIST SECTIONS

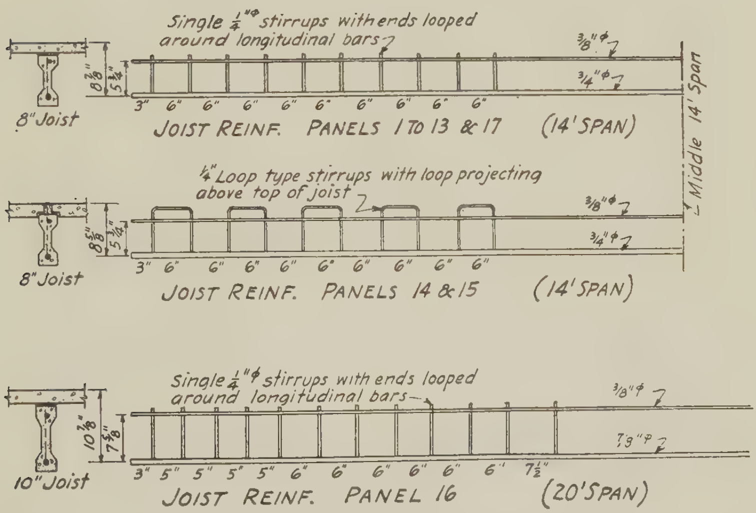


FIG. 3—JOIST REINFORCEMENT DETAILS

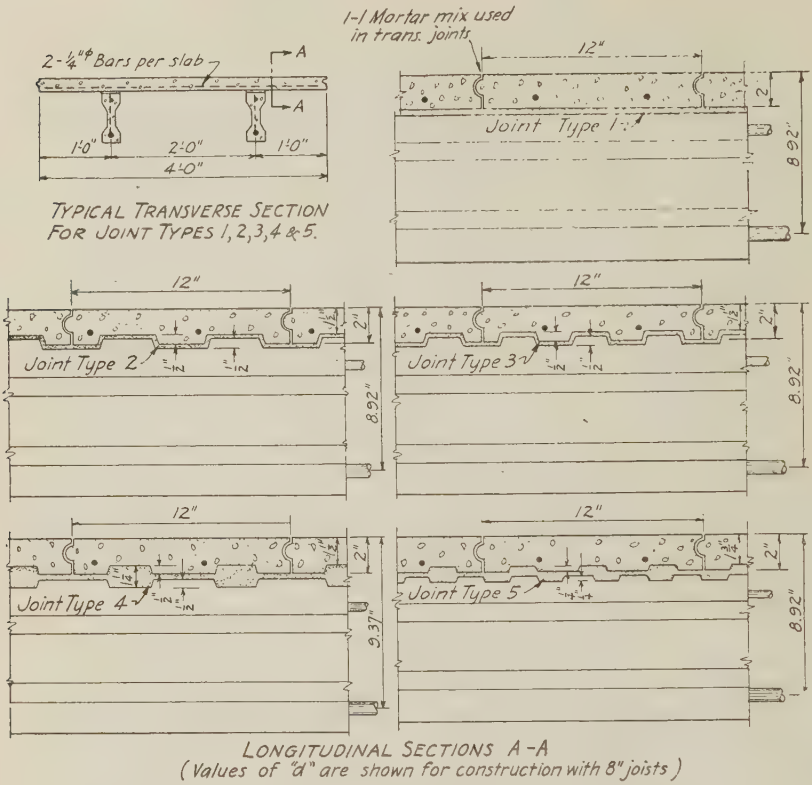


FIG. 4—DETAILS OF BOND JOINTS USED WITH 4 FT. BY 1 FT. BY 2 IN. PRECAST FLOOR SLABS

They do not require an exact or definite joist spacing. The tongue and groove joint was designed to increase the continuity and stability of the slab system during erection and until the joint mortar has hardened.

The slabs used for panels 14 and 15 were 23¼ in. wide, 28 in. long and 2 in. thick reinforced by ¼ in. bottom bars spaced about 6 in. apart. The cantilevered portions were built of half size slabs reinforced by one ¼-in. bar extending parallel to the joists. The ½-in. bars placed in the transverse joints were merely for the purpose of supporting the cantilevered portions during the test and ordinarily ¼-in. bars would be used in field practice. It should be noted that in these panels the longitudinal joint between the slab sections comes over the joist and that the joist stirrups project into the joint mortar.

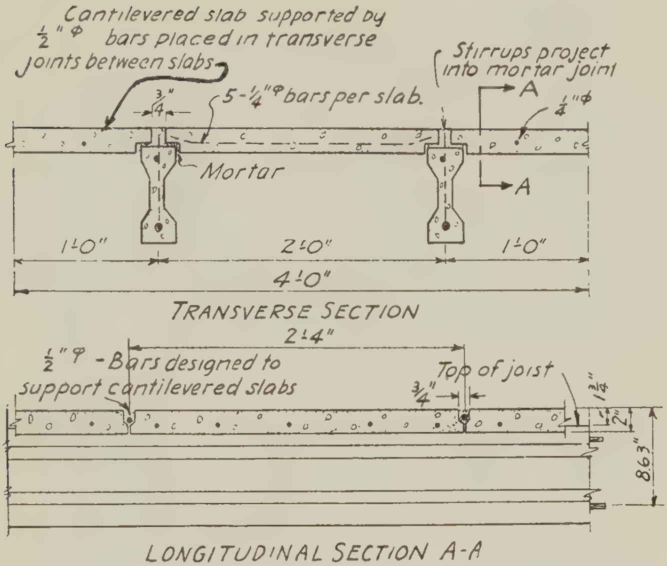


FIG. 5—DETAILS OF BOND JOINT NO. 6 USED WITH 23¼ IN. BY 28 IN. PRECAST SLABS

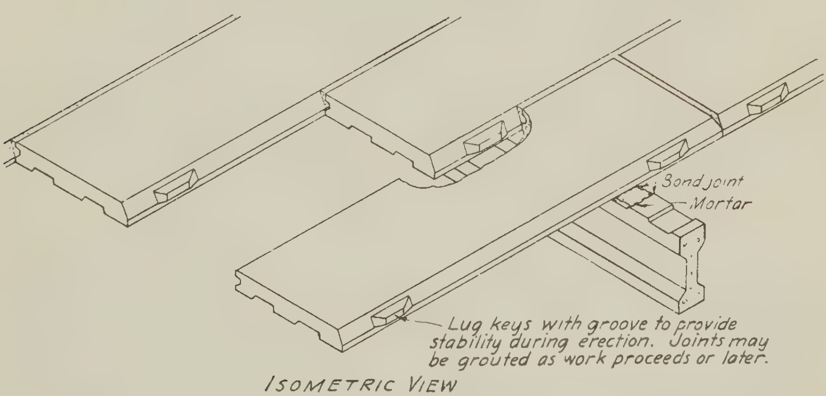


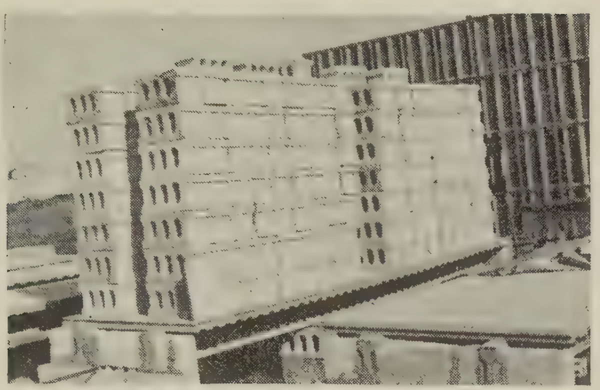
FIG. 6—SUGGESTED SLAB DETAIL FOR KEYING SLABS TOGETHER DURING ERECTION

The panels were erected when the precast members were 14 days old. The workmen were instructed to seat the slabs firmly in the mortar and thoroughly fill all joints. The mortar contact surfaces were not dampened prior to bedding. The bond joint mortar was generously spread on top of the joists as each slab was laid. The pressure exerted by the weight of the slabs facilitated the flow of the mortar to fill effectively the irregular joint spaces. The bond joints were roughly pointed with a trowel using mortar which had been squeezed from the joints. In laying the 4-ft. slabs, the transverse joints were made by first filling the grooved edge with the 1:1 mixture and then setting the slab so that the grooved edge was forced against the tongued edge of the contiguous slab previously laid. This procedure produced an excellent joint but in field operations would be considered too slow. Fig. 6 shows a type of slab joint which should provide equally good stability and facilitate erection. The joint would be filled with grout after the slabs are in place.

Texture of Mortar Contact Surfaces—The texture of the top surface of the joists and bottom surface of the slabs was smooth or rough depending on their position in the molds during casting. When the slab or joist was cast with the mortar contact surface downward or against the bottom form, the resulting surface was smooth and slightly contaminated with form oil. When cast in the top side or upward position, it was finished fairly rough and free from form oil.

Table 1 includes information regarding the bond surfaces of the individual test panels. Rough or smooth surfaces are optional with bond joint Type 1, depending on the position in casting, whereas smooth surfaces are almost compulsory with the irregular bond joints. In manufacturing practice, the surfaces should be treated to remove the form oil but this was not done in these tests.

Test Procedure—The panels were tested from 21 to 25 days after erection. Initial deflection and strain gauge readings were made and the loading of the panel with concrete block started. The block were arranged in separated tiers to prevent arching and were uniformly distributed for all panels except No. 17 where half the total load was applied at the mid-point and one-quarter the total load at each of the quarter points of the span. Subsequent sets of readings were made on all panels at load increments of 20 or 40 lb. per sq. ft. The primary purpose of the test was to evaluate the strength of the bond joint and loading usually was stopped when the bond joint failed. When the bond joint did not fail, loading was continued until the yield point of the steel was reached. This load was



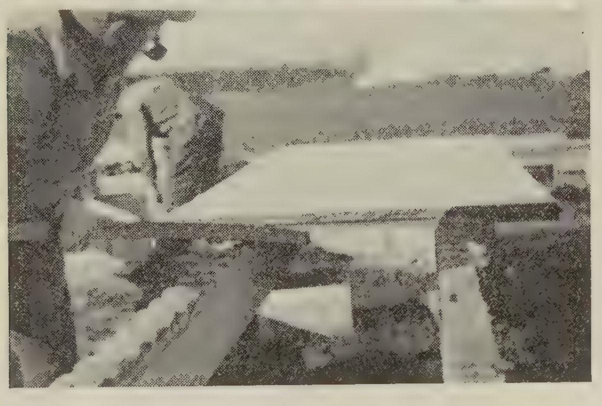


FIG. 7—PANEL 10 WITH LOAD OF 280 LB. PER SQ. FT.

FIG. 8—IN DISMANTLING THE PANELS THERE WAS FURTHER EVIDENCE OF THE BOND JOINT STRENGTH. THIS TYPE OFFERED CONSIDERABLE RESISTANCE

considered to mark the ultimate capacity of the panel. However, to get information regarding carrying capacity beyond this point, panels 4, 7, 8, 16 and 17 were loaded to complete failure. A panel under load is shown in Fig. 7.

OBSERVATIONS OF PERFORMANCE OF PANELS DURING TESTING

The test panels may be divided into two performance groups. The weaker group included panels 1 to 6 inclusive, which developed bond joint failures at comparatively low loads. The stronger group included panels 7 to 17, which, with the exception of panels 11 and 13, failed in tension and not at the bond joint.

As was expected, all panels performed similarly at loads up to fracture of the bond joint. When such failure occurred, the T-beam action was destroyed. The joists then performed as independent members and there was a sharp increase in the rate of deflection and strain with any further increase in load. The bond joint failures which occurred at loads of 100 lb. per sq. ft. or more were attended by an audible cracking sound and generally caused the joists to deflect suddenly about ¼ in. or more to bear on the emergency blocking. When the bond joint failed at lower loads as in panels 1, 2, 3 and 4, it produced a more gradual separation of slab from joist and the immediate effect was less.

Fig. 8 gives a rough idea of the strength of the irregular types of bond joints remaining after the panel test.

In panels 1 to 6 inclusive, the bond joints fractured cleanly along the contact plane between the slab sections and the mortar and as unit shear would be about equal at both top and bottom of the joint, it was indicated that the unit bond was stronger to the joist than to the slab. Panels 11 and 13 were built with irregular type bond joints and in their case joint failure sheared through the mortar keys which extended into the serrated bottom surface of the slab.

There was little difference between the various panels of each aforementioned performance group in regard to the development and extent of cracking. Fine tension cracks in the bottom flanges of the joists were first noted at loads of 40 to 60 lb. per sq. ft. Diagonal web cracks first started to develop at loads of 120 to 200 lb. per sq. ft. These cracks increased in extent and number as additional load was placed on the panel but up to the yield point of the steel, they remained fine and did not give evidence of affecting appreciably the load performance of the panel.

Test of Panel 17—The principal purpose of this test was to determine if there was an appreciable tendency of slab uplift resulting from the effect of horizontal shear in conjunction with an irregular type bond joint.

Load was applied through 2 x 4-in. loading plates at the middle and quarter points of the span in such a manner that the load did not externally restrain the majority of the individual slab units as was the case with uniform loading. In panel 17 the resistance to uplift depended mainly on the tensile strength of the joint supplemented by the weight of the slabs and the beam strength of the slab portions between the 2 x 4-in. loading plates.

In the tests of panels 1 to 16 inclusive, the only observed evidence of slab uplift at loads within the yield point stress of the steel had been

confined to panel 12 where two or three diagonal web cracks extended into the bond joint, causing a slight localized lifting of the slab from the mortar. This panel, however, sustained a load of 285 lb. per sq. ft. without structural failure of the bond joint.

Panel 17 was loaded in two stages. A total load of 7680 lb. (equivalent uniform load of 205 lb. per sq. ft.) was applied and maintained continuously for eight weeks. Following this period, the load was increased to 14,050 lb. (equivalent uniform load of 375 lb. per sq. ft.) which caused complete failure in tension. The yield point of the tension steel was reached with a load of about 11,000 lb.

The bond joint was carefully examined at intervals up to completion of the test and no fracturing, separation, slab uplift or other signs of distress in the joint were observed.

Breaking Loads of Panels—Breaking loads in the case of floor construction must be considered with a number of reservations which will be briefly mentioned later. However, the question of excess load capacity beyond the yield point of the steel sometimes arises and necessitates a test to complete failure. Consequently, it was decided to make breaking load tests on a few panels, namely, 4, 7, 8, 16 and 17.

The test of panel 17 has already been described. The four other panels had been previously loaded to the bond joint or yield point limit and the reloading to the breaking point was done some time later. The only measurement taken was the deflection with the final load.

Panel 4 had previously failed at the bond joint and in the breaking load test the joists may be considered as functioning as independent members. The total load sustained was 305 lb. per sq. ft. and the maximum deflection was about 6 in. Failure occurred by crushing of the concrete in the top flange of the joists and buckling of the ⅜-in. compression bar in each joist.

Panels 7 and 8 sustained 385 and 445 lb. per sq. ft. respectively. Maximum deflection was about 7 in. The bond joints remained intact except where the wider web cracks extended to the top of the joist and followed the joint a few inches. Both panels exhibited considerable interaction between the slab and joist up to the breaking load. When panel 7 failed, a large section of one end of a joist completely sheared from the reinforcement.

Panel 16, which was of 20 ft. span, sustained a load of 315 lb. per sq. ft. The maximum deflection was about 18 in. The bond joint functioned satisfactorily and except for short, localized fractures caused by the extended web cracks from the joist, was in good condi-

tion at the end of the test. The joists of this panel were bridged only at the supports in the same manner used for the 14 ft. spans. No lateral instability or tendency of the joists to buckle was observed.

In each of these panels some of the tension and diagonal web cracks opened to a maximum width of ⅛ in. to ¼ in. Failure of panels 7, 8 and 16 was ascribed to tension and panel 4 to compression.

There was some arching of the test loads at near the breaking point caused by the large deflections and insufficient separation between the tiers of concrete block. Arching was especially serious in the case of panel 16. For this panel a load, which had been successfully carried, promptly caused final failure after it had been rearranged to eliminate arching.

It should be noted that had the test panels been constructed on story high supporting walls as in building practice, the large deflections would have seriously endangered the stability of the walls and perhaps caused failure due to thrust or bending. It is further evident that excessive deflection of the floor system generally will produce a highly eccentric wall loading. It thus appears that the ultimate load capacity of a floor construction is not synonymous with breaking load and should be based on the structural behavior of the floor in relation to the other affected structural members.

The most significant results of these tests are considered to be in regard to the satisfactory functioning of the irregular types of bond joint under conditions of excess load and deflection and the fact that the joists, acting as independent members, sustained a load several times the allowable design load based on a T-section.

DISCUSSION OF RESULTS

The principal results of the tests are given in Table 3. There is indicated the loads which produced bond joint failure and yield point stress in the tension reinforcement. Also included is information regarding deflections and measured stresses.

The loads placed on the panels ranged from 40 lb. per sq. ft. for the panels having the weakest bond joints to 445 lb. per sq. ft., the breaking load for panel 8. Yield point stress in the tension steel was produced by a load of 200 lb. per sq. ft. in the case of panel 1 with joists functioning as independent members; by loads of 275 to 288 lb. per sq. ft. in the case of 14-ft. span panels functioning as a T-section.

Comparison of the design loads with the loads placed on panels 7 to 17 shows that these panels developed a satisfactory safety factor.

The deflections due to the weight of the slabs averaged about 0.12 in. for the 14-ft. panels and 0.20 in. for the 20-ft. panel. The deflections

TABLE 3—RESULTS OF TESTS

All panels were tested with uniformly distributed load except No. 17 where the total load was concentrated, one-half at mid-span and one-fourth at each quarter point of the span.

Loading was with concrete block arranged in separated tiers to prevent arching.

Load values refer to superimposed load; lb. per sq. ft.,

Weight of floor per sq. ft.; Panels 1, 5, 7 and 10, 25 lb.;

Panel 16, 35 lb.; all others, 32 lb.

Values of E_s and E_c assumed as follows, lb. per sq. in.; E_s = 30,000,000, E_c (Haydite Concrete) = 2,500,000; E_c (sand and gravel concrete) = 4,500,000.

Maximum load placed on panel is the highest load indicated excepting for panels 4, 7, 8, 16 and 17 which were reloaded to complete failure.

Panel No.	Loads at the Following Conditions; lb. per sq. ft.		Mid-span Deflections at Following Loads, lb. per sq. ft.; in.				Measured Stresses			
							f_s; lb. per sq. in.		f_c lb. per sq. in.	
	Yield Pt. of Tension Steel	Bond Joint Failure	40	80	160	240	80 lb. Load	160 lb. Load	80 lb. Load	160 lb. Load
1	200	40	0.18	0.43	0.98		19,000	39,000	445	740
2		40	0.17							
3		40	0.16	0.43			18,900		315	
4		40	0.13							
5		132	0.05	0.12			10,150		375	
6		160	0.02	0.05			7,950		184	
7	278	(1)	0.04	0.14	0.38	0.63	10,950	25,300	290	710
8	275	(1)	0.05	0.13	0.30	0.49	10,700	23,450	417	800
9	288	(1)	0.06	0.14	0.34	0.51	10,975	23,650	282	852
10	280	(1)	0.06	0.16	0.40	0.65	9,100	24,100	521	1,010
11	244	220 (2) 244 (3)	0.04	0.10	0.25	0.50	8,900	22,400	324	437
12	285	(1)	0.05	0.13	0.33	0.53	10,000	23,100	654	1,170
13	218	218	0.04	0.11	0.31		9,970	23,000	380	775
14	280	(1)	0.07	0.18	0.41	0.62	12,400	27,100	550	1,110
15	278	(1)	0.05	0.15	0.38	0.59	11,200	26,700	750	1,350
16	215	(1)	0.15	0.35	0.76		16,650	33,850	625	1,330
17	11,000 (4)	(1)	0.03	0.07	0.20		7,400 (5)	16,700 (5)		

(1) No failure of bond joint.
(2) and (3) South and North Joists respectively.
(4) Total load. Equivalent uniform load is 290 lb. sq. ft.
(5) Values are interpolated from load-stress curve.

due to superimposed load may be studied from the table and Fig. 9, which shows load-deflection curves for the different panels. Considering all the 14-ft. span panels, the deflections ranged as follows: 40 lb. load, 0.02 to 0.18 in.; 80 lb. load, 0.05 to 0.43 in.; 160 lb. load, 0.20 to 0.98 in.; 240 lb. load, 0.49 to 0.65 in. It should be noted that the group of 14-ft. panels built with the stronger types of bond joints, which included panels 7 to 15 and panel 17 showed much less variation in deflections.

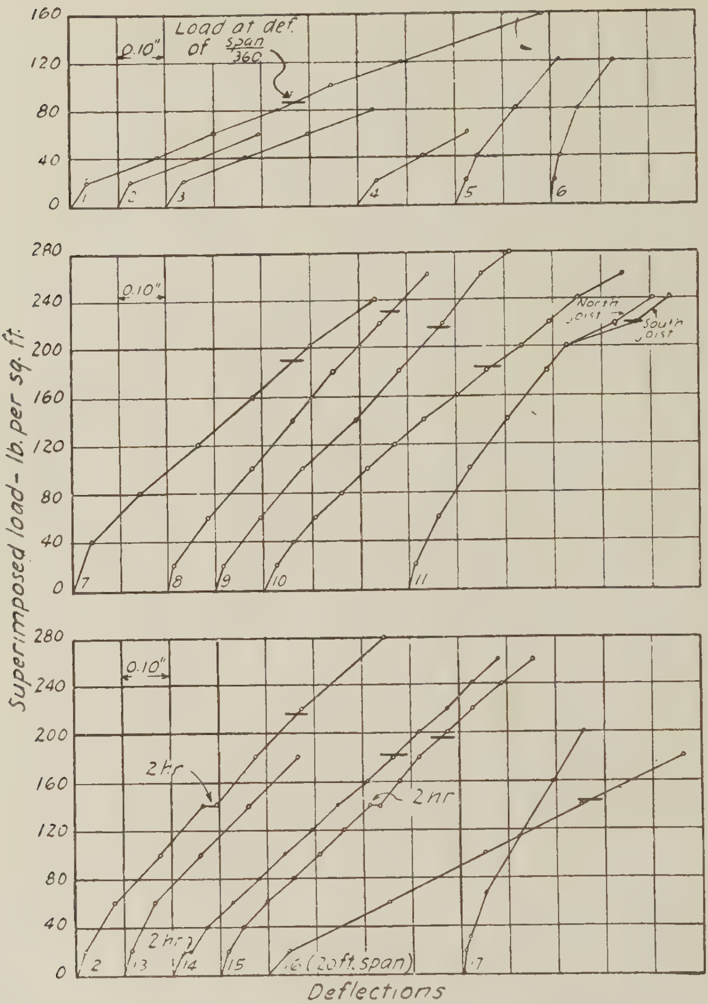


FIG. 9—LOAD DEFLECTION CURVES

The stiffest panel up to the point of bond joint failure was panel 6 for which there seems to be no adequate explanation in view that several other panels had stronger bond joints. The deflection performance of panel 1 may be considered as typical where the joists act independently of the slab. This panel deflected 0.98 in. with a load of 160 lb. per sq. ft. as compared with deflections of 0.25 to 0.41 in. for the panels functioning as T-sections. In general, the panels having

strong bond joints displayed satisfactory rigidity up to the yield point of the tension steel.

Referring to Fig. 9, it is interesting to note that with the stronger 14-ft. panels, a load of 180 lb. per sq. ft. or more is required for a deflection of $\frac{1}{360}$ x span, which is sometimes specified as a maximum value.

Panel 16, of 20 ft. span, had a lower ratio, L^4/I* than for the 14-ft. panels and, therefore, deflected more with the same unit load. For this panel a load of 142 lb. per sq. ft. caused a deflection of $\frac{1}{360}$ x span.

The measured tension stresses are in fair agreement with the computed stresses. Considering panels 7 to 13 inclusive, the measured f_s at the 160 lb. load averaged 23,570 p.s.i. as compared with the computed stress, based on the T-section, of 24,000 p.s.i. For panels 14 and 15 the average measured and computed values are respectively, 26,900 p.s.i. and 26,200 p.s.i., and for panel 16 the values are 33,850 p.s.i. and 31,400 p.s.i. These computed values are based on the T-section.

In panel 1 the joists and slab separated at a low load and it is of interest to note that the measured f_s of 39,000 p.s.i. at the 160 lb. load agrees fairly well with the computed value of 37,300 p.s.i., based on the independent joist section.

The measured concrete strains were erratic and, while undoubtedly partly due to personal and instrumental errors, the main cause is ascribed to the fact that the gauge lines crossed a slab joint, thus introducing the effect of any difference between individual joints. The measured stresses are included, however, as of possible interest for comparison with the computed values at the 160 lb. load which are as follows: panels 7 to 13 inclusive, 650 p.s.i.; panels 14 and 15, 688 p.s.i.; panel 16, 862 p.s.i. In general, the measured values exceed the computed values which seems to support, rather than weaken, the other evidences of T-beam action.

Effect of Bond Joint Design — The plain bond joint, type 1, whose shear resistance depends wholly on the adhesion of the mortar to the precast members gave the poorest results. This type joint with smooth textured mortar contact surfaces, used for panels 1, 2 and 3, failed with panel loads of 40 lb. per sq. ft.; with rough textured mortar contact surfaces, used for panels 4, 5 and 6, failure occurred with panel loads of 40 to 160 lb. per sq. ft.

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*L = span, I = moment of inertia.

Joint types 2 and 3 used with panels 7, 8, 9, 10 and 16 provide for a mechanical bond by interlocking the slab units with the joist. These joints performed very satisfactorily and were relatively stronger in panel load capacity than the tension reinforcement.

Joint types 4 and 5 differed from types 2 and 3 in that the slabs were not actually interlocked with or morticed into the joists. Mechanical bond was obtained by means of the irregular or serrated mortar contact surfaces forming a series of small mortar keys. Joint type 4 used with panel 11 fractured at a load of 220 lb. per sq. ft. Joint type 5 was relatively stronger than the tension reinforcement in panels 12 and 17 but failed at a load of 218 lb. per sq. ft. in the test of panel 13.

Joint type 6 used in panels 14 and 15 performed satisfactorily showing no signs of weakening at the maximum loads of 280 and 278 lb. per sq. ft. respectively placed on these panels.

Effect of Type of Mortar—The cement-lime mortar group include panels 3, 6, 9, 13 and 15. Comparison panels of the cement mortar group are, respectively, 2, 4, 8, 12 and 14. There was little difference in behavior between panels 2 and 3, 8 and 9, and 14 and 15. Considering panels 4 and 6, the cement-lime mortar performed better whereas in panels 12 and 13, the cement mortar gave the higher result. It should be noted that in the case of panels 4 and 6 the contact surfaces of panel 6 used with the cement-lime mortar were considerably rougher textured than the slabs of panel 4. Panels 12 and 13 were made as nearly identical as was possible except as to the mortar.

Effect of Type of Aggregate—Comparison of the Haydite concrete group consisting of panels 1, 5 and 7 with the similarly built panels 2, 4 and 8 of sand and gravel concrete reveals no consistent or appreciable difference in performance either as to deflection or load capacity. Haydite concrete panel 10 had no comparative sand and gravel concrete panel. It performed, however, in the same manner as panel 7.

Effect of Length of Span—Panel 16, constructed of 10-in. precast joists on a 20 ft. span, developed the yield point stress of the tension steel with no visible indications of distress in the bond joint. The tension stresses and ratio of deflection to span were somewhat greater than for the 14-ft. panels of similar construction. These differences are partially explained by the fact that in panel 16 the used steel area was 0.01 sq. in. less than the computed area, whereas in the 14-ft. panels the used area was 0.07 sq. in. larger than the computed area.

Effect of Type of Loading—Panel 17, with 3-point loading, performed equally as well as the similarly built panel 12 with uniform loading. Considering these tests, the effect of the 3-point loading was

no different than would be expected with an equivalent uniform load. There was no evidence of slab uplift or joint separation.

CONCLUSIONS

1. The performance of panels 7 to 17 inclusive indicates that it is possible and practicable to construct precast joist-precast slab floors in such a manner that the joist and slab function together as a T-section.

2. Considering panels 7 to 17, the minimum load causing bond joint failure was 218 lb. per sq. ft. or more than 5 times the design live load of 40 lb. per sq. ft. In 9 of the 11 panels in this group the bond joints remained effective and panel failure was due to tension.

3. Type of bond joint between the slab and joist was an important factor of panel strength and performance. The irregular type joints which provide a mechanical bond were much superior to the plain joint, type 1.

4. The shear resistance of joint type 1 was appreciably increased by using rough textured mortar contact surfaces. In general, however, the plain joint did not exhibit either satisfactory strength or dependability.

5. Bond joints 2 to 6 inclusive gave uniformly satisfactory results. Types 5 and 6 proved somewhat more practicable as to ease of manufacture and erection.

6. Considering the range of mortar strengths studied, strength of mortar was not an important factor. It does not follow, however, that a weak mortar would be satisfactory. It is believed that the mortar should be at least as strong as the 1:1:6 cement-lime mortar used.

7. Panels constructed of Haydite concrete performed similarly to comparison panels made of sand and gravel concrete.

8. Within the range of conditions studied, variations of span length and type of loading produced no appreciable effect on load performance.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June 1936. Discussion should reach the Secretary by April 1, 1936.*

Effect of Curing Temperature on the Compressive Strength of Concrete at Early Ages*

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INTRODUCTION

By this investigation Maj. John F. Conklin, District Engineer, Corps of Engineers, U. S. Army, sought definite knowledge of the effect of curing temperature on the compressive strength of concrete during the curing period stipulated in current specifications. The tests were made in the District Testing Laboratory in connection with construction work in the district.

That temperature exerts a marked effect on the hardening of portland cement concrete is well known but much remains to be learned before definite utilization can be made of the knowledge in concrete specifications. Since economy demands that the curing periods be reduced to a minimum and that forms be "stripped" as soon as possible, we need definite data so that full advantage may be taken of favorable conditions.

The most extensive data covering the effect of curing temperature on the strength of concrete are those reported in Bull. 81 of the Eng. Exp. Sta., Univ. of Ill., 1915. Since then several papers have been published on the subject of curing, among them those indicated by appended references 1 to 12.

It is not to be construed from this investigation that 1500 p.s.i. was in any way considered the optimum strength at which forms should be stripped, or curing discontinued. This value was chosen arbitrarily for comparison only. Further, it should be kept in mind that the investigation indicates only what happens to concrete near the surface of the mass and not to the mass itself. William J. Krefeld, in his paper "Incomplete Curing Weakens Concrete Surfaces," *Civil Engineering*, A. S. C. E., Vol. 3, No. 12, December, 1933, has presented an interesting and instructive picture of the effect of curing on concrete

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at different distances from the surface of the structure. His results indicate that beyond the first few inches from the surface the concrete is affected very little by external curing conditions.

DESCRIPTION AND SCOPE OF TESTS

In making these tests the aim was to expose the test specimens to exactly the same curing condition as that given the concrete in the field, and to compare these results with those obtained from companion specimens cured in the laboratory at 70° F. and 100 per cent relative humidity. Incidentally, the cylinders cured in the field received the same amount of moisture as those cured in the laboratory as the concrete was kept continuously wet. The following procedure was followed in making and storing the test cylinders:

Each day concrete was taken as placed in different parts of the form and combined in a large sample from which ten 6 x 12-in. cylinders were made. Six or eight of these cylinders were placed in the worst location on top of the wall after the pour had been completed, or were placed in special recesses in the sides of the walls and immediately boarded up. Before the recesses were covered, however, saturated burlap sacks were placed with the cylinders so that the surrounding medium could draw moisture from the burlap rather than from the specimens themselves. At the completion of a pour the top of the wall was covered with tarpaulins and steam turned on. The remaining cylinders were cured in the laboratory until time to be tested.

The cement used in these tests was somewhat coarser ground than present day normal cements, the amount retained on the No. 200 sieve being about 17 per cent; 4½ sacks per yard of concrete were used. Sand and gravel from the Ohio River was used as fine and coarse aggregate, the latter graded in two sizes: No. 4 to 1-in. and 1-in. to 2-in. These sizes were combined to give as constant a grading as possible. The proportions of the aggregates in combination, by absolute volume, were 30 per cent sand, 25 per cent large gravel and 45 per cent small gravel.

During the period that these tests were being made the mix remained the same. If for some reason it was necessary to change temporarily the mix no specimens were made until the "standard" mix had been resumed. The mix, by weight, was 1:2.54:5.85. The slump varied from 0 to 1-in. and the water-cement ratio varied according to requirements of the work.

In compliance with the specifications for winter curing, the concrete was cured for 5 days during this period (Nov., 1934 to Jan., 1935) after which the forms were generally stripped. Therefore those specimens,

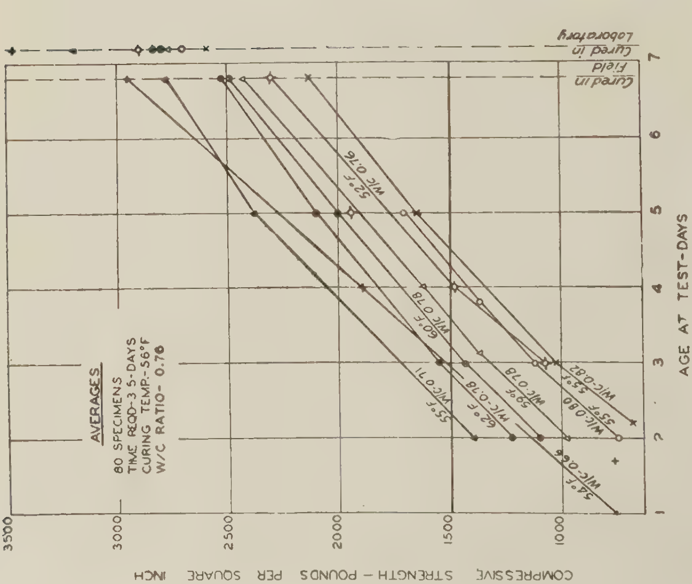

FIG. 1—TIME REQUIRED FOR SPECIMENS CURED ON TOP OF WALLS TO ATTAIN A COMPRESSIVE STRENGTH OF 1500 P. S. I.

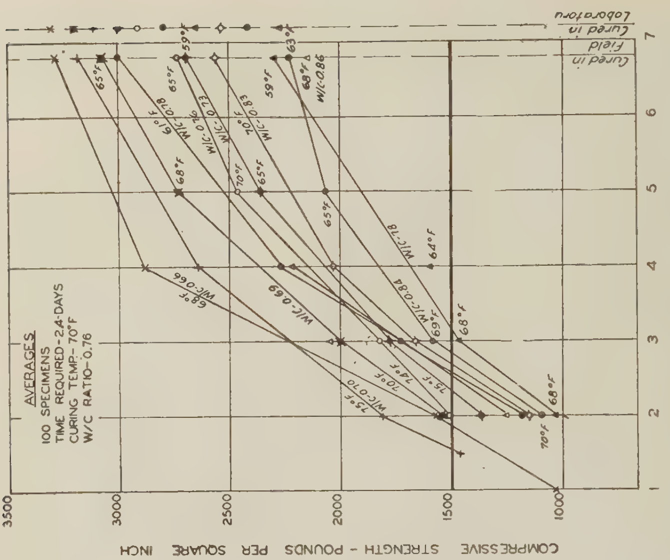

FIG. 2—TIME REQUIRED FOR SPECIMENS CURED IN SIDE OF WALLS TO ATTAIN A COMPRESSIVE STRENGTH OF 1500 P. S. I.

which were not tested at earlier ages, received the same total amount of warmth and moisture as the surface concrete of the walls for about 5 days, and were then subjected to the same weathering for the remainder of the 7-day period which was the time limit of test on any one set.

The field cured specimens were taken from their respective storage places and tested in sets of two at ages ranging from 40 hours to 7 days. The laboratory cured cylinders, companions to those cured in the field, were tested at 7 days.

Additional information obtained in connection with these tests includes initial temperature of the concrete, ambient temperature, slump and water-cement ratio. Curing temperature of the concrete in the vicinity of the specimens was obtained every four hours for the duration of the tests.

This investigation covered six weeks during the middle of the winter and 180 test specimens were made. The results were very concordant, and check closely those obtained on similar tests previously made in this District.

RESULTS

The results obtained are shown in Fig. 1, 2 and 3. Fig. 3 shows the loss in compressive strength of concrete exposed to temperatures below 70° F., the points for the curves on this chart being taken from Fig. 1 and 2. Some field cured concrete had as high, or higher temperature than that cured in the laboratory, with concordant strength results. Curves showing these results are not included in Fig. 3 since there was no loss in curing temperature or compressive strength.

The main variable affecting the results was curing temperature, the mix, brand of cement and type of aggregate remaining the same throughout, and the average of the water-cement ratios of both methods of curing being the same. This, in effect, produces the same results as though all the water ratios were the same as far as the average is concerned, leaving the curing temperature as the only real variable (see "averages" tabulated on the charts). In this connection, it is interesting to note that there were large variations in the time required to attain 1500 p.s.i. in individual instances where the curing temperatures were the same. The significance of the effect of variations in water-cement ratio on the time required to attain a certain strength is here clearly demonstrated.

Fig. 1 shows the length of time required for the concrete cured on top of the walls to attain a compressive strength of 1500 p.s.i. The averages of the two variables, water-cement ratio and curing tempera-

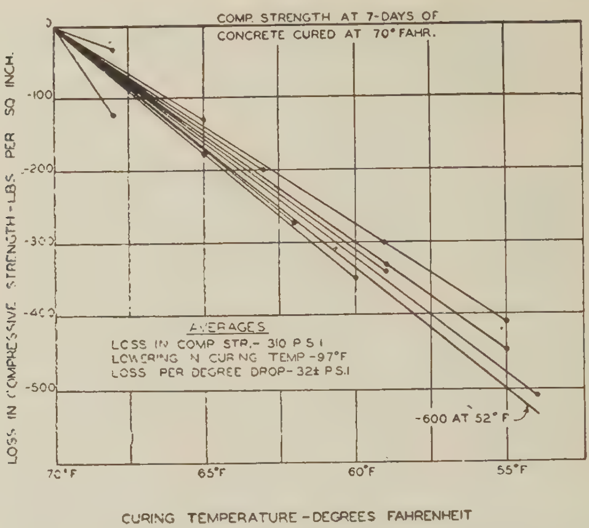

FIG. 3—LOSS IN COMPRESSIVE STRENGTH DUE TO LOWERING OF CURING TEMPERATURE BELOW 70° F.

ture, are tabulated on the chart. It will be noted that, for an average water-cement ratio of 0.76 and a curing temperature of 56° F., it took the concrete 3.5 days to attain a compressive strength of 1500 p.s.i. Fig. 2 shows the length of time required for the specimens cured in recesses in the walls to attain the same strength. In this case, the cylinders received constant warmth from the mass concrete in the wall, and were steam cured the same as the wall surfaces. With the same average water-cement ratio (0.76) and a curing temperature of 70° F., the concrete attained a strength of 1500 p.s.i. in 2.4 days. In making this comparison it must be kept in mind that the average water content of the concrete, for both conditions of curing, was the same and that the only (average) variation was in curing temperatures. Thus it is evident that the differences of 14° F. in curing temperature delayed the gain in strength for the concrete on the top of the wall 1.1 days, as compared with the concrete protected by forming. Stated in another way, the cylinders placed on top of the wall attained a compressive strength of less than 1200 p.s.i. in 2.4 days. The curves in Fig. 3 show the losses in compressive strength of that concrete subjected to a curing temperature below 70° F. during the period covered. By averaging the individual losses, it was found that the average loss was 310 p.s.i. for a lowering in temperature of 9.7° F., or about 32 p.s.i. per degree Fahrenheit. The per cent loss in compres-

sive strength was approximately 10, which means that, for the strength range obtained in these tests, the loss in strength for 7 day cylinders was approximately 1 per cent for each degree lowering of temperature; this percentage would, of course, vary with variations in compressive strength of concrete.

As before stated the 1500 p.s.i. strength requirement was arbitrarily assumed for these tests, and should not be considered as a criterion. However, if some such strength *were* specified to control the length of curing period, the results herein presented will give some idea of the variations which could be expected.

The data presented represent actual results from one series of tests indicating the effects of varying curing temperatures on the strength of concrete at ages up to 7-days. It is, of course, impossible in most instances to provide curing for more than a short period, but the decided increase in strength with increased temperature indicates the desirability of maintaining a curing temperature as high as possible, without endangering the concrete. The higher the strength obtained on the concrete surfaces the greater its resistance to wearing and weathering agencies.

Most specifications call for a minimum curing period during which time the temperature may have a range as great as 50° F. In concrete as placed the water-cement ratio will vary over a limited range. It is likely that, under these conditions there is a wide variation in quality. The most significant fact brought out in the investigation is that curing of concrete should be subjected to the closest possible control. Since water-cement ratio has such a pronounced effect on compressive strength (and all the other qualities of concrete), curing period should be longer, or the temperature be higher as the water cement ratio is increased.

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For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June 1936. Discussion should reach the Secretary by April 1, 1936.*

Concrete Slabs Reinforced with Welded Wire Fabric*

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In the discussions of the Building Code committee of the American Concrete Institute[1] several questions were raised as to the properties of cold drawn electrically welded wire fabric and its action when used as a reinforcing material in reinforced concrete slabs. It was decided to conduct a series of tests to shed light on these points. The specific questions to which attention was directed were as follows:

First, What is the effect of the spot welding upon the strength of the wire strands?

Second, Is it permissible to droop the material from the top of the slab in a region of negative moment to the bottom of the slab in a region of positive moment, instead of bending it sharply at the point of inflection or carrying a portion of the positive reinforcement straight through into the supports without bending?

Third, What is the permissible unit tensile stress in cold drawn steel wire used as a reinforcing material?

Fourth, Does the concrete encasement of the structural steel supporting members influence the effective length of the span?

PRELIMINARY TESTS

The effect of the welding process was first studied by making tension tests on a number of pieces of the wire it was proposed to use as a reinforcing material—i. e., wires of No. 3 and No. 9 W. and M. gauge. Fifty specimens of each gauge were so tested as to include one cross wire in the length under test, and for purposes of comparison, fifty more specimens of each gauge were tested between welds.

In tables below it will be observed that the strength of the wire apparently was not affected by the cross welding and in no case did the wire break in the weld. Both sizes of wire complied fully with the A. S. T. M. specifications, except that the strength of the No. 9 wire was slightly below the required 70,000 p.s.i.

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[1]Committee 501, Standard Building Code.

NO. 9 WIRE

	Breaking Strength, p.s.i.		Distance of Break from Weld in Inches
	Between Welds	Through Welds	
Highest	71,100	71,400	5
Lowest	65,200	64,400	¼
Average of 50 Specimens	67,600	67,900	1¾

NO. 3 WIRE

	Breaking Strength, p.s.i.		Distance of Break from Weld in Inches
	Between Welds	Through Welds	
Highest	83,000	91,000	7 5/16
Lowest	75,100	75,400	½
Average of 50 Specimens	78,400	78,100	3⅛

TEST SLABS

The effectiveness of the wire fabric as a reinforcing material and the influence of its positioning upon the strength of the reinforced slabs was then investigated by studying the behavior of 14 test slabs. The slabs were cast, indoors, in three groups beginning Oct. 11, 1934, and the dates of casting were so spaced as to give all slabs approximately equal time—28 days—for curing. Broken stone coarse aggregate of ¾ in. size was used, and special care was taken to secure the same proportions in each casting. All slabs were covered with burlap which was kept moist for one week. Test cylinders showed that the strength of the concrete at 28 days was between 2100 and 2400 p.s.i.

The wire fabric was furnished by the Wire Reinforcement Institute, and was shipped in rolls. Its quality has already been mentioned. When cut to length the sheets were straightened on the floor with wooden mallets, before being placed in the forms.

The fabric with which these specimens were reinforced comes normally with ten longitudinal strands, which in these cases would have necessitated a slab 40 in. wide; but in order that the results might be more nearly comparable to those conducted at another institution it was decided to make the slabs 32 in. wide and to include eight longitudinal wires. A thickness of 4 in. was chosen as representing the minimum floor thickness allowed in the usual building code. In order that the effect of length might be brought out, the supports for slabs number 1 to 7 were spaced 5 ft. apart, and for slabs number 8 to 14 the supports were on 8 ft. centers. It was desired also to investigate the effect on the slab of end restraint varying from complete

freedom to complete fixation. Hence, in each span two slabs were simply supported, one was cantilevered over one support, and 4 were cantilevered over both supports. Of these double cantilever groups the supporting steel beams of one slab of each span length were encased in concrete, as is usual in building practice.

The reinforcement was designed in accordance with the proposed New York building code, to carry a uniformly distributed live load of 150 lb. per sq. ft. The most convenient commercial fabric having the required cross sectional area contained No. 9 longitudinal wires at 4 in. centers with No. 12 transverse wires at 12 in. centers for the 5 ft. spans, and No. 3 longitudinal wires at 4 in. centers, with No. 8 transverse wires at 16 in. centers for the 8 ft. spans.

TYPES OF LOADING

Although the slabs were designed for a uniform load it was deemed advisable to test them under concentrated loading simulating as nearly as possible the moments and shears produced by uniform loading, to preclude the indeterminate arching effect of an applied uniform load. On the simply supported spans, therefore, the concentrated loads were applied at the outer quarter points of the span. In this position the shears produced are the same as those resulting from a uniform load giving the same bending moment. On the slabs having the double overhang the loads were so placed as to produce a maximum negative moment equal to twice the maximum positive moment—a condition which corresponds to complete fixation under uniform load—and so placed that the points of contra-flexure would coincide with those produced by a uniform load. On the slabs having a cantilever at one end only, the position of the loads was such as to produce equal positive and negative maximum moments, this being an intermediate state between simple support and complete fixation.

The overhang on the 8 ft. slabs, while long enough to receive the applied cantilever loads was limited by the physical conditions of the test to a length too short to produce a negative moment twice as great at the positive moment caused by its own weight, and compensating weights were hung on the ends of the slab. On the 5 ft. spans this was unnecessary, since the cantilever ends of the slabs could be made long enough to produce the required moment distribution.

All 14 slabs, with dimensions, reinforcement, and positions of load are shown in Fig. 1.

LOADING APPARATUS

In Fig. 2 is shown the apparatus by means of which the loads were applied, which may be likened in its action to a nut cracker. The

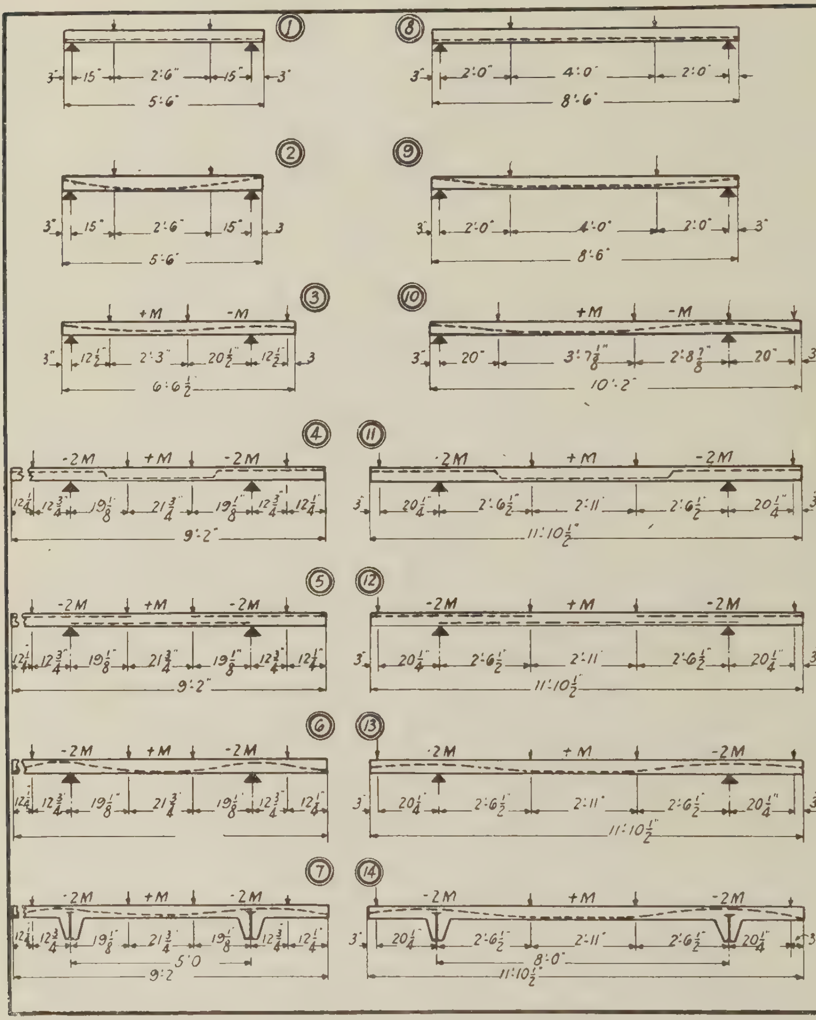

FIG. 1

load was applied by means of the testing machine to one end of the upper beam. The other end was hinged by means of a pair of vertical rods to the lower supporting beams, and these beams in turn were hung from a cross beam resting upon the weighing table of the testing machine. This made it possible to weigh accurately all applied loads, which, of course, were doubled in magnitude at the slab.

In the discussion which follows, the weight of the slab and of the beams resting upon it through which load is applied will be referred

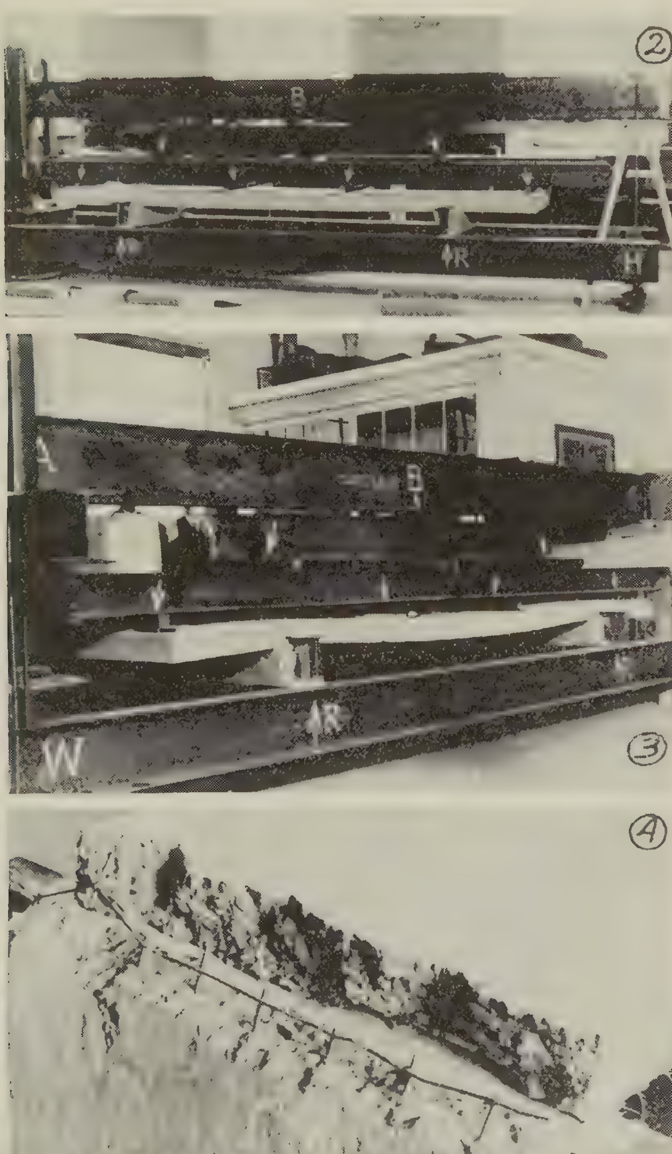

Fig. 2, 3, 4

to as the dead load. Live load is the load applied to the slab by the machine, and of course will be twice as great as indicated on the scale beam of the machine. The live load was applied in increments of 500 lb. at the machine, corresponding to 1,000 lb. at the slab. No extensometer readings were taken, but a sufficient number of deflections were read, accurate to 0.01 in., to give the contour of the slab following the application of each increment of load. Fig. 2 shows the 8 ft. double overhang slab with encased supports. This same slab is again shown in Fig. 3, carrying a live load of 18,400 lb.

TEST DATA

In all cases failure occurred by the breaking of the longitudinal wires at the point of maximum moment and in no case did any wire break in the weld. These facts are particularly significant when it is remembered that a crack always occurred at a point of maximum moment and also at this point a cross wire was always present. Thus for the longitudinal wire to break at a point away from the weld, it actually had to break away from the crack and pull out of the sound concrete. This is very clearly illustrated in Fig. 4, which shows a typical fracture. The simple slabs of course, broke near mid span, and the slabs with double overhangs, having a negative moment twice as great as the positive, naturally failed over the supports. The slabs having a single overhang both failed over the support although negative and positive moments were equal, probably due to the fact that in the region of negative moment the steel was a little closer to the compression surface than in the region of positive moment. Such a condition could easily occur, since the reinforcement tended to settle in the wet concrete. No cracks were found in the cantilever ends, although sometimes a crack occurred directly above each edge of a supporting I beam. In the slabs with cantilevers, one crack, and only one, usually occurred just outside the middle load points, although several might occur between these loads.

The behavior of the 5 ft. spans showed very clearly the influence of the strength of the concrete itself, a factor neglected in design. In the simple spans of this length only one crack formed, and the longitudinal reinforcement broke either at the load which caused the crack or when an attempt was made to apply the next increment of load. The same is true in the 5 ft. span with single overhang, where the cantilever broke off during the application of the increment following that which formed the first crack over the support. One of the double overhanging slabs of 5 ft. span also broke when an effort was made to increase the load above that which caused the first crack. In the other, the load dropped very considerably when the crack formed,

because the slab deflected away from the loading device. As the head of the machine was run further downward the slab again picked up load, but was never able to sustain the load which caused the first crack. These results indicate that the percentage of steel employed was just about the minimum amount necessary to make the cracked reinforced concrete slab equal in strength to the uncracked unreinforced slab when steel of this quality is employed.

The same phenomenon was observed in the behavior of the 8 ft. span though not in so marked a degree, for here, because of the larger percentage of steel employed, the loads carried were much in excess of those which caused the first cracks, and hence resulted in the formation of more than one crack in each slab. In general more cracks occurred in the slab with drooped reinforcement than in the others, although the difference is not pronounced except in the case of the simple spans.

The manner in which the slab cracked has an important bearing on the permissibility of drooping the reinforcement. It is evident that the resistance of concrete to tension, although neglected in design, insures the integrity of the concrete for some distance on each side of the point of contraflexure. It is in this region that the reinforcement is drooped, and hence as ordinarily constructed the reinforcement may be counted upon as being present in all regions of tension where there is danger of cracking.

RESULTS OF TESTS

As an aid in summarizing the results of these tests, Table 1 was prepared. The first column gives the slab number, the second the average measured thickness at the fracture, and the third the average measured depth to steel at the fracture. In the fourth column is given the data concerning the amount of reinforcement employed, and it will be noticed that even in the slabs containing the most reinforcement the percentage of steel present is not much more than half the amount required when steel of structural grade is used. Columns 5 and 6 give the maximum positive and negative dead load moments. Columns 7 and 8 give the maximum positive and negative live load moments due to a single increment of live load. Column 9 gives the number of increments of live load required to cause the fracture of each slab. By multiplying the live load moment for one increment by the number of increments the total live load moments given in columns 10 and 11 were found. These, added to the dead load moment, give the total moment resisted by each slab. If the slabs had been of exactly the same thickness, and if in comparable slabs the steel had been at exactly the same distance from the compression surface,

TABLE 1

Slab	Thickness	Average d	A_s	Dead Load Moment		Live Load Moment Incr'm'ts*		No. of Incr.	Total L. L. Moment		Total Mom.		Computed f_s†	
				+	—	+	—		+	—	+	—	+	—
1	4⅜	3⅞	.137°″ in 32″ width .051°″ in 12″ width —0.143%	1033	—	625	—	4.8	3,000	—	4,033	—	93,973	—
2	4½	3¾		1039	—	625	—	6.5	4,063	—	5,102	—	122,940	—
3	4	3 5/16		470	512	322	326	9 ±	2,898	2,934	3,368	3,446	91,855	93,982
4	4¼	3¼		320	647	135	266	11 +	1,485	2,926	1,805	3,573	50,139	99,250
5	4¼	3⅛		320	647	135	266	12.9	1,742	3,431	2,062	4,078	59,624	117,918
6	4¼	2 13/16		320	647	135	266	12	1,620	3,192	1,940	3,839	62,246	123,186
7	3⅞	2 11/16		304	620	135	266	13.5	1,823	3,591	2,127	4,211	71,496	141,546
8	4 7/16	3¾	.371°″ in 32″ width .139°″ in 12″ width —0.387%	2096	—	1000	—	8	8,000	—	10,096	—	93,626	—
9	4½	3 13/16		2113	—	1000	—	11.2	11,200	—	13,313	—	121.487	—
10	4¼	2½		1337	1269	516	518	11	5,676	5,698	7,013	6,967	97,516	96,876
11	4⅜	3 13/32		666	1223	218	421	18.16	3,959	7,645	4,625	8,868	47,194	90,490
12	4	2 15/16		594	1195	218	421	17	3,706	7,157	4,300	8,352	50,888	98,840
13	4	2 27/32		594	1196	218	421	14.86	3,239	6,256	3,833	7,451	46,887	91,144
14	3¾	2 7/16		546	1176	218	421	21.86	4,765	9,203	5,311	10,379	75,781	148,095

*500 lbs. applied at machine †j—.97 on 5 ft. spans; j—.93 on 8 ft. spans.

these total moments could have been used in comparing strengths. Since, however, neither of these factors was constant, the resisting capacity of each slab may be expressed in terms of the computed tensile stress developed in the steel at failure. These computed stresses are tabulated in the last two columns of the table.

It is of interest to note that slab No. 2 was apparently stronger than slab No. 1, and that the strength of slab No. 9 bore practically the same ratio to the strength of slab No. 8. Slabs No. 2 and No. 9 were reinforced with drooped steel. No special reason for this phenomenon is apparent, but it is evident that drooping the reinforcement does not impair the strength of the slab, and that up to at least 8 ft. the variation in span length produces no detrimental effect. Similarly, comparing the stresses developed in slabs No. 4, 5, and 6 it is seen again that the slab with the drooped reinforcement, No. 6, is in no wise inferior in strength to the other two. Comparing slabs No. 11, 12, and 13, slab No. 12, with separate sheets of reinforcement, developed a somewhat greater strength than the other two, but the slab with the drooped reinforcement was stronger than that in which the reinforcement was bent sharply at the points of inflection. Slabs No. 7 and No. 14, compared with the others of similar span, indicate quite forcibly the strengthening effect of the concrete haunches, and show

that it is quite reasonable to base the bending moment upon the clear spans.

The question might be raised with respect to the 5 ft. spans as to the adequacy of the reinforcement. The amount used was, as stated before, just about the percentage required to make the reinforced slab equal in strength to the unreinforced slab. Moreover, the average live load required to cause the first crack in these slabs, and incidentally to bring about the failure of the steel, was equivalent to 630 lbs. per sq. ft. From this it will be seen that the slab, even with this small reinforcement, had a factor of safety of 4.2.

SUMMARY

This series of tests demonstrates effectively that:

First: Cold drawn wire which has no definite yield point may be expected to develop its full ultimate strength in a reinforced concrete slab, and that therefore a working stress of 50 per cent of the elastic limit as defined by the A. S. T. M. specifications is quite permissible.

Second: That electrical welding of the intersections does not necessarily produce a fabric with a less tensile strength than that of the individual wires.

Third: That drooping of the reinforcement is quite as effective a method of reinforcing a slab as placing it in any other position.

Fourth: That it is quite permissible to use the clear spans in computing the bending moments.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June 1936. Discussion should reach the Secretary by April 1, 1936.*

Test of Colors For Portland Cement Mortars*

BY RAYMOND WILSON†

At the 1927 convention of the American Concrete Institute, the author presented a paper under the above title.[1] That paper reported results of exposure and strength tests on portland cement mortars colored by the admixture of finely ground pigments. Exposure tests considered in the paper had been under way for periods ranging from a few months to less than one year. The specimens have now been exposed to weather for more than 9 years. Occasional inspections have been made since the first report, though it has not been necessary to use the elaborate inspection procedure previously described.

Long-time exposure has not materially changed the import of the conclusions based on exposure for a few months. With the exception of the ultramarine blues, all pigments which did not fade within 3 to 6 months still retain their color. Some of the ultra-marine blues retain a fairly good color and others have deteriorated.

Color changes other than those brought about by fading of the pigment have assumed a relatively greater importance with lengthening time of exposure. Weathering of the pigmented cement paste has brought a greater area of aggregate into view. As discussed in the original report the effect of this aggregate exposure depends on the color of the aggregate and of the cement paste. It is extreme in such cases as a combination of a yellow aggregate and a blue pigment, which are approximately complementary colors. On the other hand the effect of exposure is small when the pigment and the aggregate are of approximately the same color, or when the aggregate is colorless or white.

Based on long periods of exposure, the major conclusions of the investigation may be re-stated as follows:

1. Mineral pigments of the types comprising the greater part of the pigments tested are not affected by exposure to weather in the presence

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†Conservation Engineer, Portland Cement Association, Chicago, Ill.
[1] *Proceedings*, Amer. Concrete Inst., Vol. 23, p. 226.

of portland cement. The more extensively used of these pigments are the various pure or impure oxides of iron, chromium oxide, and carbon black.

2. Six months appears ample to indicate the color permanence of most pigments. Ultramarine blue is a possible exception and other types of pigments not included in the tests may eventually fade after retaining their color for longer periods than 6 months.

3. The use of color-durable pigments does not insure durability of color in the mortar surface. Exposure of the aggregate may lead to a change in color of the mass. Another source of apparent color failure is efflorescence. A thin film of efflorescence deposited on the surface of colored concrete may mask the color and give the appearance of fading even though the cement paste itself has shown no change in color.

Discussion of a paper by Messrs. Ruettgers, Vidal and Wing:

"An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam"*

S. L. MEYERS†

The authors have contributed needed knowledge in a rather neglected field of concrete.

A disproportionate amount of both research work and routine testing have been on strength of concrete, while such weaknesses of concrete as porosity, (and its near relation, permeability), resistance to corrosion, volume changes, and fatigue have not received the attention they deserve.

The conclusion of the authors, "that the Boulder Dam mass concrete is so impermeable through the concrete proper that a gunite coating on the upstream face is unnecessary," seems quite justified from the results of their tests. It is also probable that the concrete in the dam will be less permeable than tests indicate; since some factors tending to make a large mass with a higher center than surface temperature and a slow flow of water with higher concentration of salts, less permeable than the specimens under test at room temperature using a more rapid flow of water with lower concentration of salts.

Not considering any chemical reactions between the water and cement during hydration, there are three stages of cement hydration depending on the amount of free water present—each stage may merge in another.

First stage: Water present in combination, absorbed, or in capillaries, no mobile water present. All four cement compounds hydrate incompletely, only tricalcium silicate hydrolyzes to some extent by setting free lime hydrate. Here the concrete corresponds to that in dam before any flow of water starts through it.

Second stage: Free water present, but saturated water containing end products not removed, resulting in equilibrium which limits hydration and hydrolysis. All four compounds liberate lime by

*Journal, Amer. Concrete Inst., March-April 1935, *Proceedings*, Vol. 31, p. 382. See also discussion Sept.-Oct. 1935, p. 125, this volume.

†Chemist, Southwestern Portland Cement Co., El Paso, Texas.

hydrolysis, (to a lesser extent small amounts of silica, alumina, and lime go into solution). Hydration more complete than in preceding stage. Corresponds to condition of concrete, (except for a thin portion upface), in dam during early stages of flow as long as water is saturated with lime.

Third stage: Soluble reactions products removed by excess water all reaction products proceed to completion, leaving no soluble lime or soluble lime compounds; but only insoluble hydrous silica, alumina, and iron. This end condition could only occur in a permeable mass of concrete after long ages of flow.

The literature cited by the authors deals mainly with small permeability test pieces and rather rapid flow of water through specimens. In such cases the readily hydrolyzed lime, above that quantity of lime precipitated as calcium carbonate, is washed out; then the excess carbonic acid dissolved in the water combines with calcium carbonate to form soluble calcium bicarbonate which is removed from the structure of concrete by the percolating water. It is this latter process, sometimes referred to as "acid corrosion," that is so common in concrete of ordinary sized structures but can only occur in large mass concrete after long periods, except in regions adjacent to cracks, or faults.

In the flow of water, containing bicarbonates and dissolved carbon dioxide, through concrete, there will come a time, as the readily available lime decreases, when the carbonate in the water will be able to precipitate all of the dissolved lime as calcium carbonate as fast as it forms to separate from solution within the concrete and the discharge water will contain no lime.

It was this condition noted by the authors who concluded "the Denver City water was found to be non-corrosive to these residual silicates; for once the early hydrolyzed lime was removed, analysis indicated no further extraction of salts by the percolating water."

Where the soluble products of a reaction are removed as formed, the reaction will go to completion; in this case lime will continue to be formed by hydrolysis and removed, either as the hydrate discharging with the water or as the carbonate precipitated within the concrete pores, but not retarding the reaction except by surface coating. In time the cement portion of the concrete will be represented by a mass of hydrous silica, alumina, and iron.

High concentration of hydroxylions will retard hydrolysis of cement, but the alkalinity required for equilibrium is over pH 12.0, and this is out of the range of all city and commercial waters and all but unusual natural waters.

A consideration of the analysis of Colorado river and Denver city water indicates the following probable reactions will occur:

Material Precipitated Within Pores of Concrete if Flow is Sufficiently Slow	Colorado River (Parts Per Million)	Denver City (Parts Per Million)
Mg $(OH)_2$	50.4	18.5
$CaCO_3$	261.0	140.0
Silica	19.0	4.5
$3CaO.Al_2O_3.3CaSO_4.31H_2O$	863.0	141.0
Fe_2O_3	0.3	0.1
Total for deposition	1193.7	304.1
Materials Present in Discharge Water		
NaCl + KCL	92.3	32.6
NaOH	67.3	1.2
KOH	8.2	
CaO	1030.0 (at 20° C)	1116.0 (at 20° C)*
Material Taken From Concrete or Used in Reactions Where Redeposited		
To form $CaCO_3$	73 CaO	39 CaO
To form $3CaO.Al_2O_3.3CaSO_4.31H_2O$	244 $C_3A.6H_2O$	40 $C_3A.6H_2O$
Lime in discharge solution	1033	1116
TOTAL	1350.0	1195.0
Total solids deposited	1193.7	304.1
Net loss of weight in concrete	156.3	890.9

*The difference in lime content of the two waters at discharge is due to difference in OH concentration.

On a basis of weight the Colorado river water will dissolve somewhat more solids from concrete than will be precipitated within the pores of the concrete; however, on the basis of volumes the condition may be reversed due to the bulkiness, or low specific gravity, of the precipitated materials, (excepting $CaCO_3$). As an example: $C_3A\ .\ 6H_2O$ has a specific gravity of 2.41, while the higher sulphate form of C_3A has a specific gravity of only 1.48. $C_3A.6H_2O$ by weight is 28.3 per cent of 863 ppm. of sulphate, but is only 17.4 per cent by volume.

Where the bulk of deposited materials is greater than the bulk of dissolved materials, pore space and therefore permeability would tend to decrease. This of course is independent of decrease of pore space due to normal hydration.

In spite of the low percentages of C_3A in the low heat cement used in Boulder Dam there will be sufficient C_3A to combine not only with the SO_3 in the cement but to form calcium-sulpho-aluminates for long ages at the indicated rate of flow. Where the volume of calcium-sulpho-aluminates formed exceeds the pore space of the concrete it will probably set up an expansive force.

The authors feel that the concrete pores may become full of deposited salts on the downstream face of the concrete, cutting off flow and building up pressure at this point. It seems likely that this condition will occur towards the center of the concrete mass, (but due to decrease

of solubility, not evaporation, in this case) because of the following reasons:

As the water comes into contact with the upper face of the dam, in the early stages, most of the chemical reactions will occur rapidly—deposition of calcium carbonate and magnesium hydroxide; the $Ca(OH)_2$ dissolved from this first zone will not (at least at first), require further $Ca(OH)_2$ to be dissolved in the central zone, but precipitates such as calcium-sulpho-aluminate which may not be formed immediately because of the low solubility of $C_3A.6H_2O$, will tend to be precipitated in the more central zones.

Both calcium hydroxide and calcium carbonate are more soluble at lower than high temperatures and some of this material will be precipitated out in the flow of water towards the central portion of the concrete mass.

Independent of the above temperature effect, cement forms super-saturated solutions from which more stable forms slowly precipitate; thus, as the super-saturated water flows from the upper face zones to the central zones deposition will occur. Pressure also tends to make substances go into solution, so that until the center of the mass becomes impermeable and the pressure at that zone as high as at the face, substances will be deposited due to decrease of pressure as they travel towards the downstream face.

In short many of the factors involved tend to deposit material at the central zones, but few, or none, to dissolve from there.

One would expect that if the authors made more tests on cements with varying fineness and chemical composition, some relationship would be shown between these conditions and permeability. It seems to be the general impression, and it is logical, that permeability decreases with increasing cement fineness. Owing to different amounts of water combined, resulting in volume differences between anhydrous and hydrated cement compounds, calculation shows a considerable difference in the void space of hardened neat pastes made from the four cement compounds.

The previous issue of this JOURNAL *announced the conclusion of this discussion in this issue. Conclusion is deferred to the Jan.-Feb.* JOURNAL *(1936), to permit the authors of the original paper to include additional data in their closure.*—EDITOR.

Discussion of a paper by R. F. Blanks and C. C. McNamara:

"Mass Concrete Tests In Large Cylinders"*

BY B. MOREELL†

In the oral presentation of this paper at the 31st Annual Convention of the Institute, Professor Carlson stated that the decrease in unit strength of the large cylinders (shown by Fig. 5) was probably the result of internal stress caused by differences of temperatures and humidities between the outer layer and the interior of the cylinder.

The authors state that "a complete explanation of the reduction in the strength of concrete due to increasing the size of the specimen has not as yet been satisfactorily developed. It is probable that the ratio of diameter of cylinder to maximum size of aggregate, internal stresses, end restraint and other factors not yet revealed combine to produce the final result."

The wide range over which the apparent compressive strength of a brittle material, such as stone and concrete, can be made to vary by changing the conditions of end restraint is not generally recognized. The magnitude of this range is dependant upon the nature of the material being tested as well as on the end conditions of the specimen. A. Föppl, in tests on 8-in. cubes, reduced the apparent compressive strength of mortar specimens to 56 per cent of the usual value by lubricating the ends of the specimen with a mixture of talc and animal fat. For granite, he reduced the apparent strength to 28 per cent of the normal value.

Mesnager tested various brittle materials in the same manner and obtained reduced strengths ranging from 85 per cent to 42 per cent of the normal values.

It appears that the resistance to lateral distention at the surface of contact between specimen and testing machine acts to increase the compressive resistance of the specimen, as does any lateral restraint. The resistance to lateral distention may be caused by friction between the contact surfaces, by deflection of the platen of the machine, by de-

*Journal, Am. Concrete Inst., Jan.-Feb. 1935; *Proceedings* Vol. 31, p. 280.
†Commander, C. E. C. U. S. Navy Bureau of Yards and Docks, Washington, D. C

flection of the edges of the specimen, or by a combination of these effects.

The large variations in strength noted above were obtained from tests on cubes. It is reasonable to assume that the effect of lateral restraint would be greater on a cube than on a standard cylinder having a height twice its diameter. Nevertheless, it is interesting to note that Gonnerman, in tests on standard concrete cylinders, obtained reductions in strength of 53 per cent by inserting sheet rubber between the platen of the machine and the specimen. (See Bulletin 14, Lewis Institute, 1925.)

While it is not stated in the paper, it is assumed that all specimens were tested in the same machine, shown in Fig. 2. It is reasonable to assume that the lateral restraint at the ends of a small specimen would be greater in such a machine than for a large specimen, the edges of which are flush with the edge of the platen of the testing machine.

In this connection, there may be some significance in the fact that for the largest size of cylinder, where the lateral restraint was probably a minimum, the reduced strength was about 84 per cent of the 6 x 12 cylinder strength, which is almost the same relationship found by the Institute's Committee 105, Reinforced Concrete Column Investigation, between the strength of concrete in the columns (where the effect of lateral restraint at the ends is dissipated in the length of the column) and the standard cylinder strength.

BY H. J. GILKEY*

Preliminary Note. The tables and figures of this discussion have been numbered to follow consecutively those of the paper. References are numbered in the body of the discussion and follow at the end of it.

INTRODUCTION

Without wishing to detract from the credit due the authors and to the Bureau of Reclamation for the admirable manner in which these tests have been planned and executed, the writer feels forced, nevertheless, to take issue with some of the most important of the conclusions offered. These major differences will be discussed first, after which brief mention will be accorded a few of the other interesting features of the tests where the writer's own experimentation and study seem to support or supplement the findings reported.

Obviously the primary purpose of this investigation was *to obtain information on mass concrete tested in large cylinders.* To round out the project by testing *other* concretes in cylinders of various sizes was logical and commendable, but to average the results from such supplementary tests with those from the mass concrete as a basis for generalizations purporting to be applicable to mass concrete is believed to

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have been illogical and hazardous and to have led to some most unfortunate manipulations and commitments. The following explanation may give an insight into the origin of the alleged interpretational fallacies.

This investigation has been necessarily complicated by the presence of two major variables: size of cylinder and maximum size of aggregate.[1] When the results from a large-aggregate mixture tested in a large cylinder are compared with those from a small-aggregate mixture tested in a small cylinder, it is observed that the latter strength is the greater just as indicated in Conclusion 18. Presumably one of the major objectives of this investigation was to determine whether the difference noted is due to:

(a) Difference in size of cylinder.
(b) Difference in maximum size of aggregate.
(c) Both the difference in size of cylinder and size of aggregate.

It is obvious that such a situation calls for the exercise of unusual care in the analysis of results to make sure that differences due to one of these factors are not mistakenly attributed to the other. It is in this respect that the analysis of these data is believed to be at fault, with the result that some unfortunate statements have entered the record under auspices that assure them a longevity that will be measured in decades.

Referring to Fig. 3, to which the major points in this discussion relate, the procedure has been to average all of the test results for each size of cylinder (regardless of size of aggregate) and to plot the mean curve obtained thereby. The mean curve of Fig. 3 is then reduced to a percentage and plotted as the appealing generalization of Fig. 4, which is then used as the basis for "correcting" the data plotted on subsequent figures, thus closing the circle.

This procedure, *instead of separating the variables, has blended them.* The downward trend from left to right indicates the combined effect of size-of-cylinder and size-of-aggregate. Even this is not a true indication, however, because of the differences in numbers of specimens of the different mixtures along the different size ordinates.[2], [3] And in

Size of Spec.	2x4	3x6	6x12	8x16	12x24	18x36	24x48	36x72	Totals
Max. Aggr. ⅜	12	12	14	14	6	6	—	—	64
¾	—	12	13	12	8	8	6	—	59
1½	—	—	155	12	9	9	9	3	197
3	—	—	—	—	6	6	6	3	21
6	—	—	—	—	—	—	14	3	17
9	—	—	—	—	—	—	—	13	13
Totals	12	24	182	38	29	29	35	22	371

[1]A third major variable, amount of coarse aggregate, is also present, but in these tests it always accompanies the size-of-aggregate variable and need not be considered separately at this time. It does receive brief consideration later in the discussion.

[2]The writer is indebted to the authors for a list of numbers of specimens (presumably in Series II). This list is summarized in the following tabulation:

[3]Note, for example, the pronounced downward trend that could be obtained for the mass concrete curves of Fig. 22 by the same process of vertical averaging.

addition to all this, no line has been drawn between *mass concrete* and *ordinary concrete* or *mortar*. For example the value for the 24 by 48 in. specimen is an average value for aggregates ranging from 6 in. to ¾ in. maximum while the 18 by 36 in. diameter ranges from 3 in. to ⅜ in. maximum.

Obviously the large aggregates could not extend to the small sizes of specimens and for economic reasons the small aggregates were not extended to include the largest sizes of specimens. The result (Fig. 4, neat, compact and appealing), is a curve of *corrections* for *mass concrete*, evolved from mortars at the left, cobbles at the right and an assortment of pebbles and cobbles in between that would gladden the heart of a geologist, but which do not clarify the problems of *mass concrete*. Fig. 4 is entitled "Effect of Size of Test Cylinder on the Compressive Strength of Concrete." It is suggested that "Assorted Variables" be substituted for "Size of Test Cylinder."

Fortunately those portions of the data relating to *mass concrete* are consistent and convincing, once they are isolated and studied (one variable at a time). The following assertions are the result of such study and constitute the primary theme for discussion.

1. Within the range of *mass concrete* mixtures (maximum aggregate 3 in. or over) there is no evidence whatever in support of Conclusion 1 that "The indicated strength . . . decreases as the size of specimen is increased."

2. All of the *mass concrete* evidence of this investigation is in direct opposition to Conclusion 4 that "Maximum size of aggregate has no appreciable effect on the compressive strength of concrete, etc."

3. To add to the confusion the authors have "compounded their felony" by evolving, by the averaging process and from the questionable *small-aggregate* evidence of Series II,[4] a curve of correction factors (Fig. 4) for use with *mass concrete* and have, with admirable self-assurance, proceeded to "correct" the data of their subsequent figures (Fig. 6, 8, 11, 17 and possibly others) for "size-of-cylinder effect," prior to plotting them. Under these conditions such figures, at least, may be expected to support the conclusions offered.

Briefly then the main contentions are: that conclusions purportedly applicable to mass concrete have been based entirely upon trends not apparent in the mass concrete tests; that size-of-cylinder effect and size-of-aggregate effect have been interchanged for the mass concrete and probably for concrete in general; that data from Series I, bearing upon questions for which Series II was intended to supply the answers,

[4]Which may or may not be further complicated by the "slight adjustments" in fine sand and cement mentioned at the top of page 285.

seem to have been disregarded entirely in the formulation of conclusions; and finally that the findings have been complicated by the introduction of questionable correction factors to the evidence prior to plotting.

DISCUSSION—SIZE OF CYLINDER AND SIZE OF AGGREGATE

From the introductory statements, it is apparent that Series II shows trends for the *mass concrete* portion of the evidence that are at variance with those for the concretes and mortars with maximum aggregates 1½ in. or under. To determine then just what is shown calls for an unscrambling of the evidence.

In Fig. 22, all of the *experimentally determined* evidence of Fig. 3 (data of Series II, Table 6) that relates to the *mass concrete* is plotted above and all that relates to *ordinary concretes and mortars*, is plotted below. The following points stand out:

1. For the *mass concrete* there is no significant indication of a size-of-cylinder effect.

2. For the *mass concrete* there is pronounced and consistent evidence of size-of-aggregate effect at both 28 days and 90 days. In *every* case the larger the aggregate, the lower the strength.

3. Within the range of *ordinary concretes* and *mortars* there appears to be a pronounced reduction in strength with increased diameter of cylinder up to 18 in., above which the trend appears either to cease or, strangely enough, to reverse itself.[5], [6]

4. On the whole the lower figure supports Conclusion 4 as applied to the *ordinary concretes* and *mortars*.[7] Certainly there are no pronounced size-of-aggregate effects and such as there are, are in the

[5]In spite of the apparent consistency in the indicated size-of-cylinder effect within this range of concrete, it is felt that a definite commitment on this point is not justified on account of:
(a) The lack of confirmation from other tests, either of this investigation or from other published researches.
(b) The lack of a similar indication within the field of *mass concrete* either for the tests of this series or those of Series I (Tables 5 and 9 and Figs. 26 and 27).
(c) The ceasing of the trend and even possible reversal at the 18 inch diameter size without apparent reason.
(d) The fact that these data are so definitely at variance with the other evidence of these and other tests on the subject of size-of-aggregate effect, which point will be amplified as the discussion progresses.

[6]One point that may bear on apparent size-of-cylinder effects within the smaller ranges of aggregate and specimen sizes is the probable fact that the amount of puddling per unit of material, as the mold is filled, increases as the size of specimen decreases. The subconscious over-working of small specimens may well be expected to result in some added loss of water. if not in increased strength from better placement. Such effects however, can hardly be attributed to the size of cylinder, even though they may frequently accompany differences in cylinder size.

[7]In spite of the apparent consistency of the indication, it is felt that its validity as a basis for generalization (even for these mixtures) is open to serious question. Reasons are as follows:
(a) The presence of such pronounced and definite size-of-aggregate effects in the mass concrete tests of the same series.
(b) The presence of remarkably consistent size-of-aggregate effects for the 1½-in. and ¾-in. maximum aggregates of Series I of these tests (Tables 5 and 9 and Fig. 26 and 28) for 3 water-cement ratios each at 4 test ages and all in the same size of specimen. While Series I was designed primarily to cover the subject of wet screening, none of these results are from wet-screened material, and they appear to bear directly upon the point in question. The fact that the authors seem to have ignored the Series I results completely and accepted the opposing indications of Series II, apparently without a question, was due perhaps more to oversight than intent. Certainly there is nothing in the paper to introduce a question regarding their validity.
(c) The considerable volume of published data that indicate important size-of-aggregate effects over the entire range of aggregate sizes extending from the sands up into the cobbles.

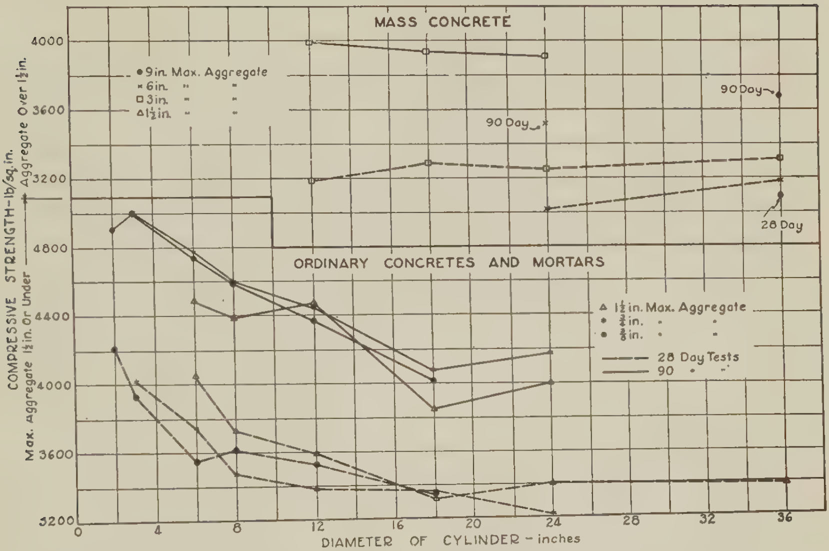

FIG. 22—STRENGTH VS. SIZE OF CYLINDER, SERIES II, TABLE 6. SAME DATA AS FIG. 3 EXCEPT FOR THE OMISSION OF THE "ESTIMATED VALUES." ALL OF THE MASS CONCRETE DATA APPEAR IN THE UPPER PART OF THE FIGURE

main reversed between 28 days and 90 days. The 90-day results are more in line with the other evidence.

Table 9 is a rearrangement of the data of Table 5 (Series I). The strength ratios of Col. j, k, l, m, show the consistent and pronounced size-of-aggregate effects over the wide range of water-cement ratios and ages covered. It will be noted that about half of these tests were on wet-screened material and that the wet-screened specimens with 1½ in. aggregate have been used as the basis for comparison. This is consistent with practice.

Fig. 23 shows a typical size-of-aggregate reduction curve from some of the data of Series I. Naturally this curve (although for Series I tests instead of Series II) does not differ greatly from the "size-of-cylinder" reduction curve of Fig. 4, which is not strange in view of the fact that Fig. 4 is probably influenced more by size-of-aggregate than by size-of-cylinder as was pointed out in the introduction.

Fig. 26 is a good basis for over-all study of much of the evidence of Series I (Tables 5 and 9). Effects of size of cylinder, size of aggregate, and wet screening are all shown, including specimens in which the aggregate exceeded one-fourth the diameter of the cylinder. Table 9, Col. (f), (g), (h), (i) should be used, however, if comparisons are de-

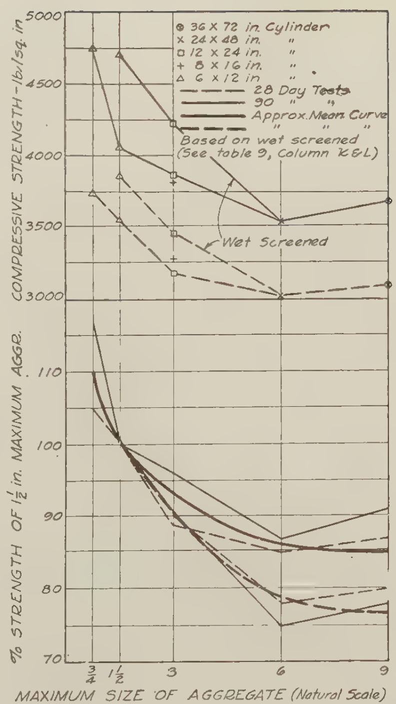

FIG. 23—EFFECT OF SIZE OF AGGREGATE, SERIES I, TABLE 5. SPECIMENS B 1 - B 5 AT 28 AND 90 DAYS

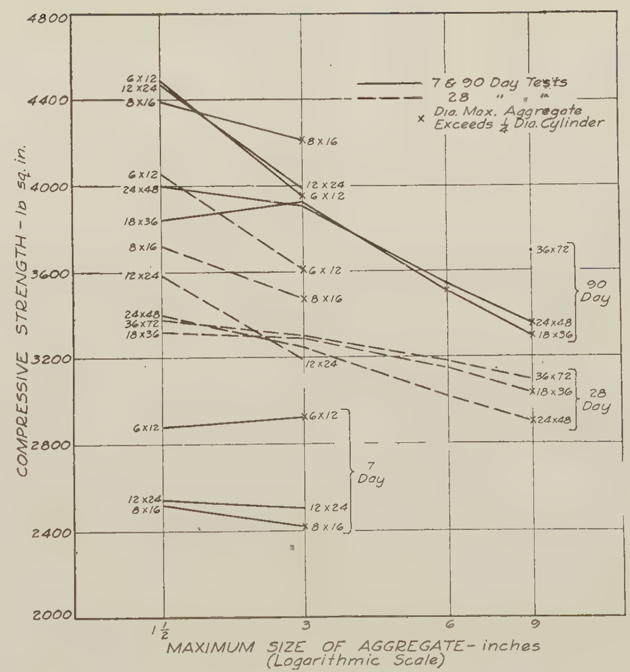

FIG. 24—COMPRESSIVE STRENGTH VS. MAXIMUM SIZE OF AGGREGATE WITHIN THE PRACTICAL SIZE RANGE FOR DIFFERENT SIZES OF CYLINDERS, SERIES II, TABLE 6. GROUPS B 9 AND B 10 OMITTED

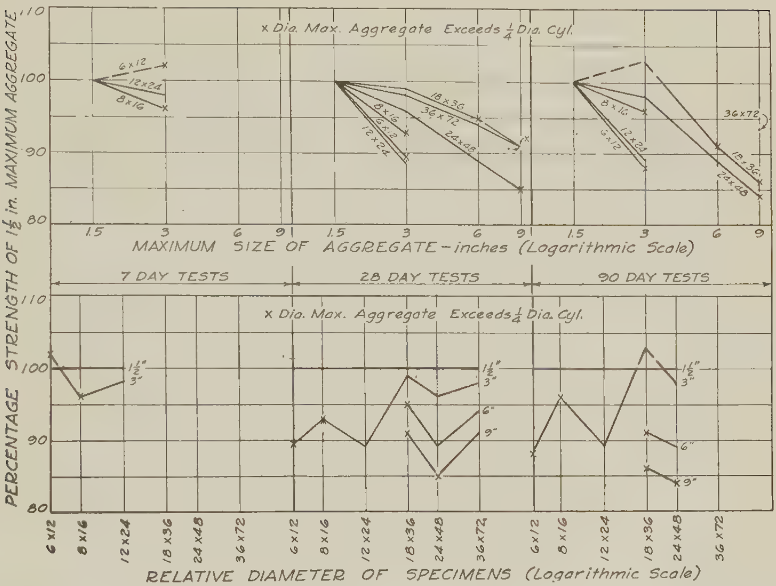

FIG. 25—RELATIVE STRENGTH FOR DIFFERENT SIZES OF AGGREGATES AND CYLINDERS AT AGES OF 7, 28 AND 90 DAYS, SERIES II, TABLE 6. GROUPS B 9 AND B 10 OMITTED. COMPLETE OTHERWISE

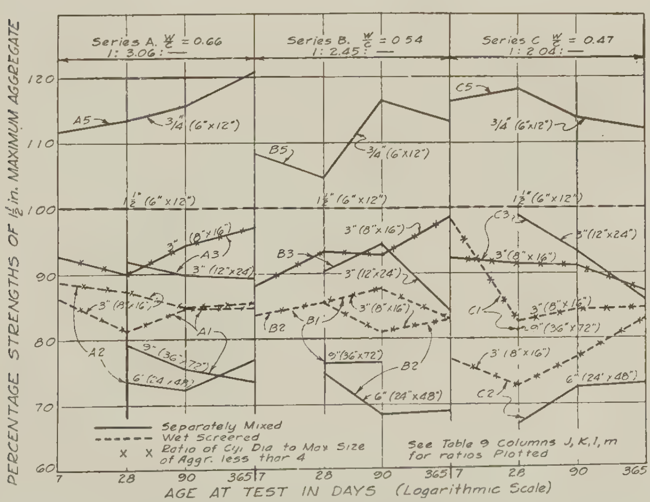

FIG. 26—RELATIVE STRENGTHS FOR DIFFERENT SIZES OF AGGREGATE AND SPECIMENS FOR 3 WATER-CEMENT RATIOS AT 4 AGES OF TEST, SERIES I, TABLE 5

sired between the material which was wet screened and that which was not, for 1½ in. aggregate.

For clear-cut evidence from unscreened mixtures of Series I on both the "size-of-cylinder effect" and the "size-of-aggregate effect" this entire investigation offers nothing more conclusive or definite than the Fig. 27 and 28.

Fig. 25 shows in other arrangements the consistent size-of-aggregate effect and the lack of a consistent size-of-specimen effect for all of the data of Series II (Tables 6 and 10) for all aggregates 1½ in. maximum size or larger. It should be noted especially that in both parts of the figure sizes of aggregate always fall in sequence; sizes of specimens never do.

EVIDENCE FROM OTHER RESEARCHES ON EFFECT OF SIZE OF SPECIMEN

Among the few tests on record there are still fewer in which there were not variations in the manner of fabrication, curing, treatment of the ends, machines and speeds of testing, etc., that limit the dependability of the findings. The most authoritative of the tests are probably those by Gonnerman (1) from which he states that "lower strengths were generally obtained with the larger cylinders" (1 p. 250 No. 3). He states further, however, that "the decrease in strength with size of cylinder, was not important for diameters of 6 in. or less" whereas it is in exactly that range that the greatest size effect is apparent in the data of Series II.

The writer has failed to find consistent indications of size effect in his own tests in which many comparisons have been made between 6 by 12 in., 3 by 6 in. and 2 by 4 in. cylinders, but none of which have been in the mass concrete range. A few representative data have been published (2 p. 434, 440, 442, 525). More often than not either the 6 by 12 in. or 2 by 4 in. specimens occupy the middle position as regards strength. Unpublished tests made within the past year on mortar specimens give the following results:

Size of Spec.	Age at Test	No. Tests Averaged	Compr. Str.
2x4	7 da.	14	2650
3x6	"	11	2800
2x4	28 da.	96	5350
3x6	"	61	5520
6x12	"	7	5400

The utmost care was taken to exclude variables other than size of specimen. All specimens were tested on the same Southwark-Emery

(1) See references at end of this discussion.

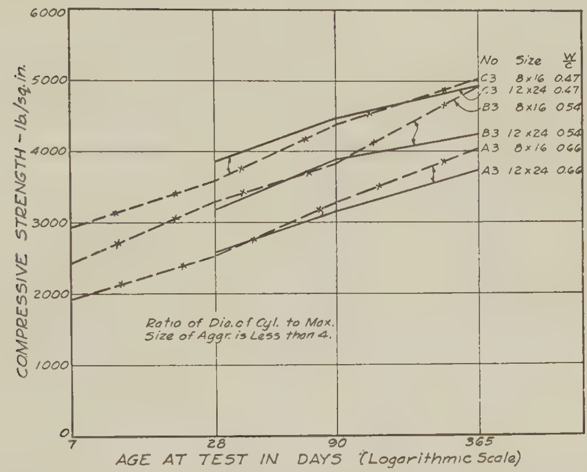

FIG. 27—8 BY 16 IN. AND 12 BY 24 IN. CYLINDERS COMPARED AT 3 TEST AGES AND 3 WATER-CEMENT RATIOS, SERIES I, TABLE 5. 3 IN. MAXIMUM AGGREGATE IN ALL SPECIMENS

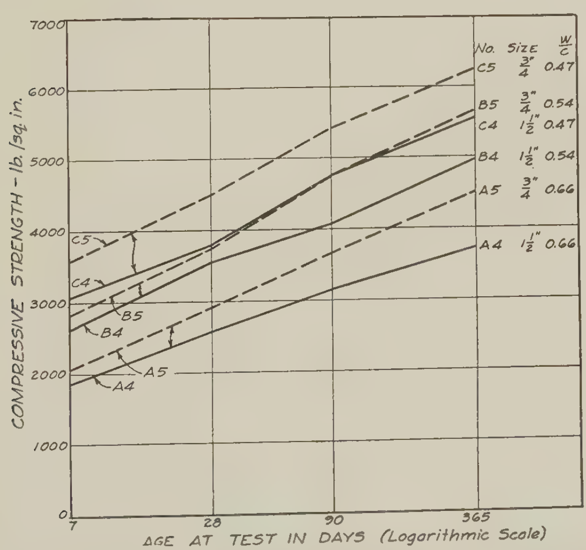

FIG. 28—1½ IN. AND ¾ IN. AGGREGATE COMPARED AT 4 TEST AGES AND 3 WATER-CEMENT RATIOS, SERIES I, TABLE 5. ALL SPECIMENS 6 BY 12 IN.

testing machine, with appropriate sizes of spherical bearing block and testing speeds.

Among the tests which would seem to indicate most clearly a consistent size-of-cylinder effect are those made in connection with the construction of the Santeetlah Dam (3 p. 5). The results from these specimens cannot be accepted at their face value, however. They were field-fabricated and later shipped to Washington for test. They constituted a first venture into the realm of large specimens and there were present many disturbing factors, unavoidable under the circumstances. The following excerpt from a Bureau of Standards report issued in August, 1928, covers this aspect briefly: "Even though in all probability the difference in strength of the small and larger cylinders may be assigned in great part to different fabrication and curing conditions and thus invalidate the tests as a measure of the respective strengths of small size and large size cylinders made of the same material and cloud the influence of difference in aggregate size, nevertheless, the tests have considerable value."

OTHER RESEARCHES ON SIZE OF AGGREGATE

Tests by Dwyer and Bates (4) illustrate definite size of aggregate effects over a selected group of mixtures, as was pointed out in discussion (4 p. 626). Tests by Ira L. Collier (5 p. 731) show, for his workable mixtures, consistent increases in strength as the coarseness of the aggregate was decreased by substituting sand for gravel. Also they show decreasing strength for increasing coarseness of sand (Fig. 8) except for two of the wet, harsh mixtures, which probably segregated badly with an accompanying reduction in the actual water-cement ratio as placed.

In a discussion of a report by F. R. McMillan (6 p. 522) are cited pertinent data on the effect of size of aggregate, all of which bear out what are believed to be the dependable portions of the author's findings.

That the reductions in strength continue with increased coarseness of sand in a mortar, if the water-cement ratio be held constant, is indicated on Fig. 9 of (7 p. 379). The finest sand at the left (100-48) gave a mortar too stiff for proper placement at the water-cement ratio used, accounting for the low strength for the finest grading. Other pertinent data appear in the same paper and also in the discussion of it. The work of Powers at Bull Run (8 p. 388), shows the same decrease of strength with increase in aggregate size. Powers' findings are reinforced in his closure (8 p. 430).

THE EFFECT OF VARIATIONS IN THE AMOUNT OF COARSE AGGREGATE

Little mention has been made of this factor, but in these tests it is

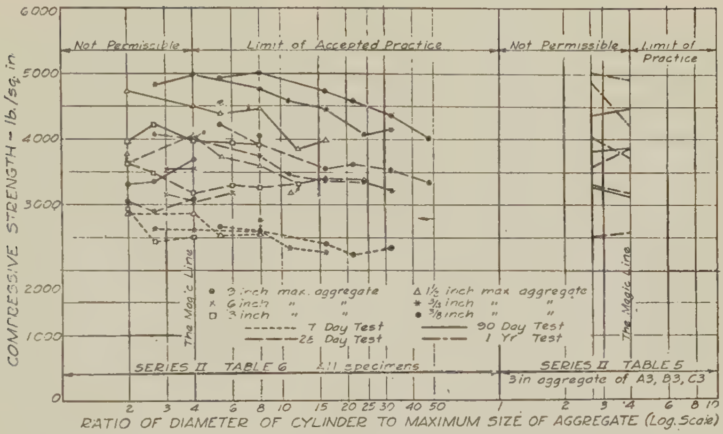

FIG. 29—STRENGTH VS. RATIO OF DIAMETER OF SPECIMEN TO MAXIMUM SIZE OF AGGREGATE

present and inseparable from the size-of-aggregate effect. In practice there are few instances in which it is necessary or desirable to alter materially the amount of coarse aggregate except as the grading (size) changes. The larger the maximum size used the greater is the amount that may be incorporated, as these tests so well demonstrate.

Referring to Table 9, Group A, column b, it will be noted that the amount of coarse aggregate varies from 8.81 parts at a 9 in. maximum to 4.50 parts at a ¾ in. maximum size. The mortar is constant for a group. Fundamentally it is of interest to know to what extent the variations attributed to size may have been due in part to amount. Practically, or at least for present purposes, it is not essential that these two variables be separated. (4 Fig. 6), (6 p. 524).

On the basis of other studies, however, it may be stated with reasonable certainty that it is the size and not the quantity of coarse aggregate that accounts for the major portion of the variation in strength that is attributable to the Siamese pair of variables present in this paper wherever the maximum size of coarse aggregate is mentioned.

DISRUPTING INFLUENCE FROM VOLUME CHANGES

Were the size of specimens to be increased indefinitely a point would be reached at which there would be important size effects because of the disrupting volume changes that occur (a) from the mechanical effects of crystal growth, (b) from the stored chemical heat of setting. If the curing were other than standard there would also be (c) the volume changes and distortions that accompany changes in moisture content. In these tests factor (c) was not present. That (a) and (b) had no

TABLE 9—SEE SERIES I, TABLE 5 OF PAPER FOR SUPPLEMENTARY DATA ON PROPORTIONS, WATER-CEMENT, RATIOS, ETC.

Line	No.	Crs. Aggr. Parts By Wt.	Crs. Aggr. Max. Size	Cylinder Size	Cylinder Ratio To Ag.	Compr. Strengths 7 Day	Compr. Strengths 28 Day	Compr. Strengths 90 Day	Compr. Strengths 1 Yr.	Strength Ratios 7 Day	Strength Ratios 28 Day	Strength Ratios 90 Day	Strength Ratios 1 Yr.	Line
Col.	a	b	c	d	e	f	g	h	i	j	k	l	m	
1	A1	8.81	9	36x72	4.00	—	2600	3120	3570	—	0.79	0.76	0.74	1
2		6.62[1]	3	8x16	2.67	1890	2670	3500	4110	0.86	0.81	0.85	0.85	2
3		5.43[1]	1½	6x12	4.00	2190	3290	4120	4840	1.00	1.00	1.00	1.00	3
4	A2	8.08	6	24x48	4.00	—	2290	2890	3700	—	0.73	0.73	0.77	4
5		6.62[1]	3	8x16	2.67	1850	2720	3380	4110	0.89	0.87	0.85	0.86	5
6		5.43[1]	1½	6x12	4.00	2080	3120	3980	4800	1.00	1.00	1.00	1.00	6
7	A3	6.62	3	12x24	4.00	—	2560	3130	3720	—	0.92	0.90	0.90	7
8		6.62	3	8x16	2.67	1910	2510	3280	4050	0.93	0.90	0.94	0.97	8
9		5.43[1]	1½	6x12	4.00	2060	2790	3490	4160	1.00	1.00	1.00	1.00	9
10	A4	5.43	1½	6x12	4.00	1830	2570	3150	3720	1.00	1.00	1.00	1.00	10
11	A5	4.50	¾	6x12	8.00	2050	2920	3650	4510	1.12	1.14	1.16	1.21	11
12	B1	7.05	9	36x72	4.00	—	3090	3680	—	—	0.76	0.76	—	12
13[2]		5.30[1]	3	8x16	2.67	2420	3470	4220	4960	0.84	0.86	0.88	0.83	13
14[2]		4.35[1]	1½	6x12	4.00	2880	4050	4810	5970	1.00	1.00	1.00	1.00	14
15[3]	B2	6.47	6	24x48	4.00	—	3020	3540	4110	—	0.75	0.68	0.69	15
16[2]		5.30[1]	3	8x16	2.67	2420	3470	4220	4960	0.84	0.86	0.81	0.83	16
17[2]		4.35[1]	1½	6x12	4.00	2880	4050	5180	5970	1.00	1.00	1.00	1.00	17
18	B3	5.50	3	12x24	4.00	—	3180	3880	4210	—	0.90	0.94	0.84	18
19		5.30	3	8x16	2.67	2410	3290	3830	4910	0.88	0.93	0.93	0.99	19
20		4.35	1½	6x12	4.00	2740	3520	4110	4980	1.00	1.00	1.00	1.00	20
21	B4	4.35	1½	6x12	4.00	2600	3560	4060	4990	1.00	1.00	1.00	1.00	21
22	B5	3.60	¾	6x12	8.00	2810	3740	4730	5670	1.09	1.05	1.17	1.14	22
23	C1	5.88	9	36x72	4.00	—	3740	—	—	—	0.82	—	—	23
24		4.41[1]	3	8x16	2.67	2980	3790	4640	5200	0.99	0.83	0.84	0.85	24
25		3.62[1]	1½	6x12	4.00	3030	4590	5500	6140	1.00	1.00	1.00	1.00	25
26	C2	5.39	6	24x48	4.00	—	3370	3980	4650	—	0.67	0.72	0.73	26
27		4.41[1]	3	8x16	2.67	2810	3670	4250	5310	0.77	0.73	0.77	0.83	27
28		3.62[1]	1½	6x12	4.00	3660	5040	5500	6390	1.00	1.00	1.00	1.00	28
29	C3	4.41	3	12x24	4.00	—	3860	4490	4930	—	0.99	0.93	0.86	29
30		4.41	3	8x16	2.67	2920	3590	4380	5030	0.93	0.92	0.91	0.87	30
31		3.62	1½	6x12	4.00	3160	3920	4810	5770	1.00	1.00	1.00	1.00	31
32	C4	3.62	1½	6x12	4.00	3060	3800	4760	5560	1.00	1.00	1.00	1.00	32
33	C5	3.00	¾	6x12	8.00	3560	4500	5440	6260	1.17	1.19	1.14	1.12	33

[1]Col. b. Nominal coarse aggregate content after wet screening.

[2]Lines 13 and 14 and 16 and 17. Strengths identical except Col. h of 14 and 17. Probably a clerical error in transcription of data.

[3]Line 15, B2. 24x48 appears to be repeated in Table 6 as B6, or Table 10 Line 6.

significant influence on the strengths seems to be indicated by the lack of any apparent reduction in strength for any mixture above the 18 in. size. Yet the 36 by 72-in. cylinders had a mass 8 times that of 18 by 36 in. specimens. Moreover the smaller the aggregate the greater weakening from the volume changes that accompany size since each of the three types of disrupting influences mentioned increases with the cement content of the concrete. The 36 by 72-in. cylinders of 1½ in. maximum aggregate of Series II had more than twice the cement content of the same size cylinder with 9 in. maximum aggregate. Yet

TABLE 10—BASED ON TABLE 6 OF PAPER. TESTS OF SERIES II INCLUDES ALL RESULTS FROM CYLINDERS OVER 3 IN. AND MAXIMUM AGGREGATES OVER ¾ IN.

Line	No.	Crs. Aggr.		Cylinder		Compr. Str.			Str. Ratios			Line
		Parts By Wt.	Max. Size	Size	Ratio To Ag.	7 Day	28 Day	90 Day	7 Day	28 Day	90 Day	
	a	b	c	d	e	f	g	h	i	j	k	
1	B1	7.05	9	36x72	4.00	——	3090	3680	——	0.91	——	1
2	B6	6.41	6	"	6.00	——	3180	——	——	0.94	——	2
3	B7	4.93	3	"	12.00	——	3310	——	——	0.98	——	3
4	B8	3.30	1½	"	24.00	——	3380	——	——	1.00	——	4
5	B1	7.05	9	24x48	2.67	——	2900	3340	——	0.85	0.84	5
6[1]	B6	6.41	6	"	4.00	——	3020	3540	——	0.89	0.89	6
7	B7	4.93	3	"	8.00	——	3250	3910	——	0.96	0.98	7
8	B8	3.30	1½	"	16.00	——	3400	4000	——	1.00	1.00	8
9	B1	7.05	9	18x36	2.00	——	3030	3300	——	0.91	0.86	9
10	B6	6.41	6	"	3.00	——	3140	3510	——	0.95	0.91	10
11	B7	4.93	3	"	6.00	——	3300	3940	——	0.99	1.03	11
12	B8	3.30	1½	"	12.00	——	3320	3840	——	1.00	1.00	12
13	B7	4.93	3	12x24	4.00	2500	3190	3990	0.98	0.89	0.89	13
14	B8	3.30	1½	"	8.00	2540	3590	4470	1.00	1.00	1.00	14
15	B7	4.93	3	8x16	2.67	2420	3470	4220	0.96	0.93	0.96	15
16	B8	3.30	1½	"	5.33	2530	3730	4390	1.00	1.00	1.00	16
17	B7	4.93	3	6x12	2.00	2930	3610	3960	1.02	0.89	0.88	17
18	B8	3.30	1½	"	4.00	2880	4050	4490	1.00	1.00	1.00	18

[1]Line 6. B6, 24x48 appears to be same as B2 of Table 5 (or of Table 9 Line 15).

the 36 by 72 in. specimens with 1½ in. aggregate gave strengths that averaged slightly higher than those for the 18 by 36 in. specimens. It seems safe to conclude therefore that in no instance was volume change or cement concentration a significant item in these tests.

EFFECT OF WET SCREENING ON STRENGTH AND SLUMP

A restudy of the data (Tables 5 and 9) or a reference to Fig. 23 will show that the wet-screened specimens of Series I gave strengths consistently above those of the specimens separately proportioned. This might be due to loss of more moisture than cement in the screening process, to grinding or other action from the added working that the wet-screened material received, or to some other factor, as yet unrecognized. Here is a problem for further study because wet screening seems destined to play an increasingly important role in the inspection of mass concrete.

Without challenging the probable validity of Conclusion 16, as applied to these tests, there are unquestionably limitations to its applicability. A mixture becomes sloppy indeed after the coarse aggregate has been extracted and in the writer's experiences the concrete became increasingly fluid as successive increments of coarse aggregate were removed. If the constant slump is due to the action of the cobbles in the mixer, as stated in Conclusion No. 17, then it

would seem that the cobbles must introduce a considerable grinding action that might well account also for the increased strength that the wet-screened specimens did possess, the last sentence of Conclusion No. 17, to the contrary, notwithstanding.

From the outline for Series II as given in Table 4 and the descriptive text at the bottom of p. 282 it appears that the proportions used for mixtures B6-B10 inclusive (Series II reported in Table 6) were selected from preliminary wet screening tests on B1, but that all of the specimens of Series II were from batches weighed out and mixed without any wet screening being done on them. Certainly Table 6 gives no indication of any wet screening tests[8]. Nevertheless Fig. 16, 17 and 18 show plotted strengths from wet screening tests which are referred to Table 6. It is important to know, therefore, whether the strengths reported in Table 6 do or do not represent actual wet screening tests. If they do, more information should be given regarding such factors as how the proportions of the wet-screened material were determined. If they do not represent results of actual wet screening tests it is incumbent upon the authors to indicate how they have found it possible to present plotted compressive strength *results* for *wet-screened concrete that was not wet screened*[9]. Perhaps the authors will, while on the subject, indicate more fully the technique by which they were able to determine slumps on mixtures that included 3, 6 and 9-in. cobbles respectively. Both Tables 5 and 6 report slumps for mixtures which contained aggregates of these sizes.

It is suggested that the legend of Fig. 18 might well be changed to read "Effect of *Size of Aggregate* on the Modulus of Elasticity of Concrete." It will be noted also that Fig. 17 is one of those for which "the data were reduced to the 36 by 72-in. cylinder basis."

Assuming concurrence in the points raised, reference to size of test cylinder may be deleted from Conclusion No. 7. The last sentence of Conclusion 17 should be deleted or altered and entire rewording is suggested for Conclusion No. 15 in accordance with the previous discussion.

YOUNG'S MODULUS

Referring to Conclusions 11 and 12, it may be that the tests supply more significant evidence on the effect of maximum size of aggregate

[8]The facts that definite proportions are given for each mixture instead of a wet screening indication as in Table 5, and also that the indicated slumps are not in accord with the constant values reported for wet-screened material both lead to the inference that Table 6 contains no specimens which were actually made from wet-screened material.

[9]The writer questions also the validity of the "slight adjustments" mentioned at the top of page 285. Are the authors certain that there should not also have been a slight adjustment for water lost in the wet screening process or for grinding action in the mixer or for other things? Such "slight adjustments" may account for some sizable differences as is the case in those for "size-of-cylinder effect." Certainly the "slight adjustments" were "least slight" for the small aggregate mixtures of Series II which are so strikingly out of line with the other evidence offered.

on the elastic properties of concrete than the authors had realized when they phrased Conclusion 11. The data from Table 6, Fig. 5 and also Fig. 18 while showing no consistent variation in modulus of elasticity with diameter of cylinder (and if the earlier discussion is valid, none should be expected), does show a rather definite increase in Young's modulus with increases in the maximum size of coarse aggregate used. This trend is the reverse of that for strength. The writer plotted all of the reported ultimate strengths against moduli of elasticity as in Fig. 11 and found that for each successively larger maximum aggregate, the ultimate strength was reduced and the stiffness was increased. The plotted points fell in rather wide overlapping zones, but there was no question about the increased steepness of the mean line for each larger maximum size of aggregate. The following explanation seems to be reasonable and adequate:

In all of these tests, as the maximum size of aggregate was increased the amount was also increased. The aggregate is stiffer than the mortar and the concrete, to an increasing extent, was influenced by the stiffness of the aggregate. It must be kept in mind, of course, that the modulus of elasticity is an elastic property (measuring stiffness, not elasticity) and is measured over a range of loading in which the specimen acts as an elasticunit.

On the other hand the ultimate strength is not an elastic property. The increasing maximum size of aggregate particles progressively lowers its ultimate resistance with the result that lines of increasing steepness and of decreasing horizontal projection are necessarily produced as the maximum size of aggregate increases, other factors being constant.

Generally speaking, Conclusion 12 is correct, and is in accord with many published observations, most of which date from the work of Walker (9). A tentative qualification might be added to cover the secondary reversal in trend that these tests show to be present when the combined variable of maximum size and quantity of aggregate is present.

POISSON'S RATIO

Hearty support is offered for Conclusions 2 and 14 for specimens within the writer's range of direct observation; namely, 2 by 4 in., 3 by 6 in. and 6 by 12 in. cylinders. From a great many tests on many different mixtures, both of concretes and their mortars, no consistent trend of variation has been discovered for Poisson's ratio. The results of some of these tests have been recorded. (2 p. 484-7; 536). A fair average value for Poisson's ratio for any concrete seems to be 0.19 or 0.20, with extreme values rarely beyond 0.15 and 0.25, all of which

observations are in excellent agreement with the data of this paper. All of the data of recent years on Poisson's ratio for concrete seem to indicate that many of the values reported from earlier tests (10 p. 666) are too low which probably indicates some loss in the measurement of lateral displacement.

RATIO OF MAXIMUM SIZE OF AGGREGATE TO DIAMETER OF SPECIMEN

On page 287 an allusion is made to "Past experience, etc.," as establishing the efficacy of the magic factor of 4. While the writer concedes that 4 is a good lower limit to set from the standpoint of placement, he would, nevertheless, be interested in knowing something of the extent to which its selection was actually based upon definite information that smaller ratios of diameter to aggregate size produce unsatisfactory test results.

Specifications regularly contain the one-fourth clause (and properly so, no doubt) and occasionally one sees in the literature statements that "It has been shown to be good practice to limit, etc.," or "Tests have shown, etc.," (11 p. 16) but the writer does not recall having seen any record of tests designed to cover this point specifically.

To obtain further information all of the results of this paper were plotted with strengths against diameter-aggregate ratios as shown on Fig. 29. There is certainly neither evidence of weakening nor of decreasing uniformity. Nor do the curves of Figs. 24, 25 or 26 and 27 supply such an indication. Of course these results are averages of unstated numbers of individual tests, but if there was a serious decrease in the uniformity of the individual tests it should show as a weakening of the average since there is no reason for upper limits of strength to exceed those obtainable under conditions which are presumably more favorable.

Before leaving Fig. 29 it is of interest to digress sufficiently to point out the high degree to which it supplies a bird's-eye view of Series II, Table 6. The decided upward trend to the left of the curves for 3/8 and 3/4 in. maximum size of aggregates shows very clearly the evidence which, when plotted on Fig. 3 led the authors to their "alleged erroneous conclusion" relative to the effect of size of specimen on strength as applied to mass concrete. This same trend reappears at each test age and supplies something of the same general cast to Fig. 29 as it did to Fig. 3. The figure also shows very clearly the successive stepping down in strength at each test age as the maximum size of aggregate increases. In pondering Fig. 29 it should be kept in mind that horizontal readings are ratios and not diameters of cylinders, although proportional to them for any one maximum size of aggregate.

The following revised wording is suggested for Conclusion 3:

3. While the ratio of 4 as the lowest permissible limit for the ratio of diameter of mold to maximum size of aggregate, is conservative and proper, there is no sustained evidence from these tests that values as low as 2 produce concrete that is any weaker or less uniform than that for ratios ranging from 4 to 48.

SUMMARY OF DISCUSSION

1. Question is raised whether the present state of our concrete knowledge justifies a definite commitment on the subject of size of cylinder vs. strength of concrete.

2. In the data of these tests there is no evidence of size-of-cylinder effect within the range of mass concrete mixtures (aggregate larger than 1½ in.) or for specimens of ordinary concrete which are more than 12 in. or at most 18 in. in diameter. (Fig. 22, 24, 25, 26, 27.)

3. Within the range of mass concrete mixtures these data supply consistent indications that the maximum size of aggregate has a definite and important influence on the strength of the concrete and that for other conditions constant, the strength is reduced as the maximum size of aggregate is increased. (Fig. 22, 23, 24, 25, 26, 28 and Tables 9 and 10.)

4. Tests of Series II recorded on the last 20 lines of Table 6, which fall entirely without the range of mass concrete mixtures, give indications of size-of-specimen effect and little indication of size-of-aggregate effect.[10] Because these indications are out of line, both with the mass concrete test portions of the same series and also with the results of Series I (see Table 5 and ratios of Table 9) and with records of other tests (to some of which references are given) these indications must be considered questionable as applied to concretes and mortars within any size-of-aggregate range, and certainly they do not constitute a proper basis for generalizations or correction factors applicable to mass concrete.

5. From the above it follows that the author's Conclusions 1 and 4 are contrary to their own evidence and that Fig. 3 and 4 are invalid and misleading. Moreover, Fig. 6, 8, 11, 17 and possibly others have been made invalid by virtue of applied "corrections" obtained from Fig. 4.

6. Within the range of 2 by 4 in., 3 by 6 in. and 6 by 12-in. cylinders hearty agreement has been found with the authors' Conclusions 2 and 14 over a wide range of mixtures, materials and test ages.

[10]At the top of page 285 mention is made of "slight adjustments" by the addition of "fine sand and a little cement." The validity of these alterations is questioned, and they may well account in part for some of the "out-of-line" results of these particular tests.

7. With the omission of the last clause of Conclusion 7, as an unnecessary restriction, agreement is recorded with the rather obvious Conclusions 5 and 7.

8. Study of these and other data seem to indicate that the strength of the concrete is not greatly influenced by variations in the amount of aggregate used, but that the stiffness (modulus of elasticity) increases with the amount of aggregate but is probably independent of the grading. This conclusion supplements the authors' Conclusion 11 and introduces a modifying factor of secondary importance to Conclusions 12 and 13.

9. Within the size and cement factor range of these tests and for the standard curing condition, there is no evidence of reduction in strength from volume change effects.

10. Contrary to Conclusions 15 and 17 the wet screening did seem to increase the compressive strength. (See Tables 5 and 9). Reasons for this are not apparent, but perhaps the added working in the presence of the coarse aggregate introduced significant grinding action[11] that decreased the average size of aggregate particle, thereby increasing the strength. This would be in line with the recorded constant slump as successive sizes were screened out. Experience with hand-mixed concretes within the ordinary 1½ or 2 in. aggregate range show increasing fluidity, as successive sizes are removed, and both slumps and strengths that were in excellent agreement with re-proportioned concretes of the same grading. Further studies of wet-screening seem to be desirable.

11. Question is raised regarding the wet screening tests of Fig. 16, 17 and 18 which are referred to Table 6 (Series II) in which there were presumably no wet screening tests other than preliminary ones to determine what proportions of materials should be used. There is questioned also the validity of altering the reported proportions of Series II by the addition of "small amounts of fine sand and cement" as reported at the top of page 285. This may or may not have been a disturbing variable. Such attempts to "make adjustments" do complicate rather than safely simplify. Question is also raised as to the technique employed in obtaining slumps for concretes that contained 3, 6 and 9-in. cobbles.

12. While there is nothing but hearty agreement with the practice of limiting the maximum size of aggregate to ¼ the diameter of the specimen, the data of these tests give no indication of added lack of uniformity or weakening when this ratio reaches values as high as ½.

[11]The five minute mixing time used increases the possible importance of grinding action.

Nor is added strength or uniformity in evidence as the ratio decreases to values as small as 1/48. (Fig. 29, 26).

CLOSING REMARKS

In spite of the strong exception taken to some of the most important of the conclusions reached from these tests, the only disagreement with the supporting data lies in a range of small aggregates and relatively small specimens. In general the investigation has been so well planned and executed that the data secured supply what seems to be an adequate basis for conclusions regarding a number of the most important questions relating to mass concrete. The experimental (as distinguished from interpretational) differences lie entirely within the realm of small-aggregate mixtures susceptible of ready checking in any competently staffed and normally equipped concrete laboratory. The great contribution from these tests has been in the field of large cylinders and large aggregates and it is believed that when the results secured in this field are permitted to speak for themselves, they will stand as a monumental and unimpeachable piece of concrete research, a credit to the Bureau of Reclamation and a testimonial to the diligence and competence of its staff.

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AUTHORS' CLOSURE

EXPLANATION AND GENERAL COMMENT

The major part of Prof. Gilkey's discussion has apparently been inspired as a result of: first, a misconception of the term, "mass concrete," and the principal reasons for performing the series of tests reported, and second, an erroneous interpretation of the detail data presented in Table 6 showing the effect of size of test cylinder on compressive strength. The authors assume full responsibility for these misunderstandings and deeply regret that the complicated nature of the tests precluded a detailed presentation of the methods and procedure employed and the complete analyses of data in a paper for publication in the JOURNAL. The complete information constitutes a volume of no small proportions which is at present in the process of perparation for publication in the form of a Government Bulletin.

Mass concrete as used in the paper under discussion refers to any concrete, regardless of the size of aggregate, placed in structures of massive proportions. It is true that cobbles up to six or nine inches in diameter are usually (not always) used in mass concrete but this does not necessarily mean that the terms "large aggregate concrete" and "mass concrete" are synonymous.

One of the primary purposes of the tests reported was the evaluation of the effect of maximum size of aggregate on various properties of concrete, as shown by Tables 3 and 5. Obviously, the size of aggregate could not be varied without also varying some other conditions which were chosen on the basis of practical considerations. In other words, water-cement ratio and slump were maintained constant and the mix proportions, or cement content, varied accordingly. To further maintain constant test conditions, the cement-sand ratios for a given set were held the same. It is also obvious that all the mixes included in the Series I tests could not be tested in 36- by 72-in. cylinders for economic reasons. The last fact necessitated that certain mixes be tested in various sizes of cylinder, as indicated in Tables 4 and 6, to provide direct experimental factors for converting the test values given in Table 5 to a comparable basis, i.e., the same size of test specimen.

The second primary object of the tests was to establish the relationship between mass concrete containing large aggregate and the same concrete after wet-screening to eliminate the larger particles. Concrete containing large aggregate requires large test cylinders while it is usually customary, in field control work, to eliminate the cobbles

TABLE 11—EFFECT OF SIZE OF TEST CYLINDER ON THE COMPRESSIVE STRENGTH OF CONCRETE

Boulder Dam Aggregate

These strengths were taken from Table 6

Age	Mix No.	Max. Size Agg., In.	Cylinder Sizes, Unit Strengths, and Relative Strengths															
			2 by 4 In.		3 by 6 In.		6 by 12 In.		8 by 16 In.		12 by 24 In.		18 by 36 In.		24 by 48 In.		36 by 72 In.	
			Str.	%	Str.	%	Str.	%	Str.	%	Str.	%	Str.	%	Str.	%	Str.	%
7 days	B-7	3	—	—	—	—	2930	100.0	2450	83.6	2500	85.3	—	—	—	—	—	—
	B-8	1½	—	—	2860	99.3	2880	100.0	2530	87.8	2540	88.2	—	—	—	—	—	—
	B-9	¾	2610	101.6	2600	101.2	2570	100.0	2330	90.6	2260	87.9	—	—	—	—	—	—
	B-10	⅜	2640	110.0	2600	108.3	2400	100.0	2210	92.1	2320	96.7	—	—	—	—	—	—
	Avg., %, 6 by 12			105.8		102.9		100.0		88.5		89.5						
28 days	B-1	9	—	—	—	—	—	—	—	—	—	—	3030	100.0	2900	95.7	3090	102.0
	B-6	6	—	—	—	—	—	—	—	—	—	—	3140	100.0	3020	96.2	3180	101.3
	B-7	3	—	—	—	—	3610	109.4	3450	104.5	3190	96.7	3300	100.0	3250	98.5	3310	100.3
	B-8	1½	—	—	3650	109.9	4050	122.0	3730	112.3	3590	108.1	3320	100.0	3400	102.4	3350	101.8
	B-9	¾	4060	120.8	4020	119.6	3750	111.6	3470	103.3	3380	100.6	3360	100.0	3210	95.5	—	—
	B-10	⅜	4210	126.0	3930	117.6	3550	106.3	3610	108.1	3530	105.7	3340	100.0	—	—	—	—
	Avg., %, 18 by 36			123.4		115.7		112.4		107.0		102.8		100.0		97.7		101.3
	Avg., %, 6 by 12			109.8		102.9		100.0		95.2		91.5		89.0		86.9		90.2
90 days	B-1	9	—	—	—	—	—	—	—	—	—	—	3300	100.0	3340	101.2	3680	111.5
	B-6	6	—	—	—	—	—	—	—	—	—	—	3510	100.0	3540	100.9	—	—
	B-7	3	—	—	—	—	3960	100.5	4180	106.1	3990	101.3	3940	100.0	3910	99.2	—	—
	B-8	1½	—	—	4710	122.6	4720	122.8	4390	114.3	4470	116.4	3840	100.0	4000	104.2	—	—
	B-9	¾	4830	118.7	5000	122.9	4790	117.7	4600	113.0	4450	109.3	4070	100.0	4170	102.5	—	—
	B-10	⅜	4910	122.4	5000	124.7	4740	118.2	4590	114.4	4370	109.0	4010	100.0	—	—	—	—
	Avg., %, 18 by 36			120.6		123.4		114.8		112.0		109.0		100.0		101.6		111.5
	Avg., %, 6 by 12			105.0		107.5		100.0		97.5		94.9		87.1		88.5		97.1
All Ages	Avg. % of Str. in 6 by 12[a]			106.9		104.4		100.0		93.7		92.0		88.0		87.6		91.6

[a]These values are weighted averages of the average values for individual ages

by wet-screening and to test the remaining portion of the mix in small, 6- by 12-in. cylinders. Thus it will be seen that the relationship sought involved a number of variable factors including size of test cylinder, maximum size of aggregate, cement content, and sand-gravel ratio. The test data presented in Table 6 evaluate the effect of wet-screening under two conditions, i.e., when the resulting concrete is tested in the same size of cylinder as the full mix, and when the resulting concrete is tested in various sizes of specimens. Table 5 includes data for directly comparing full-mix concrete tested in appropriate sizes of test specimens and the corresponding wet-screened concretes tested in 6- by 12- in. cylinders.

Briefly summarized, the tests under discussion provide data for evaluating the effects of three major variables on various properties of concretes: maximum size of aggregate and wet-screening from choice and, size of test cylinder from necessity in order to interpret intelligently the relationships found. The variable factors were extended over a sufficient range to establish properly the fundamental trends or principles involved and the data obtained led to certain conclusions as stated in the original paper, published in the January-February 1935 issue of the JOURNAL. Since that time, confirming and supplementary tests have been completed with an entirely different source of aggregate and with independent mixes. The original conclusions are well sustained and additional pertinent and interesting information has been obtained as outlined in the following paragraphs.

EFFECT OF SIZE OF TEST CYLINDER

A somewhat different analysis of the data presented in Table 6 is given in detail in Table 11. All the strengths for each mix have been expressed as relative strengths, in per cent, for the various sizes of cylinders and only such relative values used to obtain the averages. Additional tests made with aggregates from the Grand Coulee dam site also furnish a direct comparison of strengths for the same concrete in 6- by 12- and 24- by 48-in. cylinders, as shown in Table 12.

The re-analysis of data from the Boulder Dam tests, the additional test results from the Grand Coulee studies and Professor Gilkey's analysis from Figure 22 are all summarized in comparison with the original analysis in Table 13. It is most gratifying that the four sets of values indicate substantially the same trend and provide ample proof af the validity of the curve shown in Figure 4.

The average values listed in Table 13 as well as the individual curves shown in Fig. 3 and 22 indicate that regardless of the size of aggregate (¾ to 9 in.), approximately the same strengths for a given

concrete are obtained in 18 by 36, 24 by 48, and 36 by 72-in. cylinders. This fact greatly simplifies and reduces the cost of testing mass concrete, as has already been pointed out in previous publications.[1]

End restraint has been studied both theoretically and experimentally as a possible cause of the lower strengths found in large cylinders. However, these studies did not reveal any new relations or any new theories that might be accepted as a satisfactory explanation. At present it seems most probable that the reduction is closely related to the so-called "skin effect," or differential stresses within the test specimens.

TABLE 12—EFFECT OF SIZE OF TEST CYLINDER ON THE COMPRESSIVE STRENGTH OF CONCRETE

Mix by Wt.	W/C by Wt.	Slump in.	Weight lb./c.f.	Cement bbl./c.y.	Water lb./c.y.	28-Day Str.—p.s.i. 24"x48"	6"x12"	Ratio
1:6.75	.66	7½	150.6	1.29	320	2440	2900	0.84
1:6.86	.66	10	150.3	1.44	358	2320	2510	0.92
1:9.40	.67	2⅜	156.8	1.02	257	2610	2910	0.90
1:5.86	.57	6⅛	151.7	1.47	315	3240	3660	0.89
1:5.18	.57	10	150.8	1.60	343	3060	3300	0.93
1:8.32	.58	1⅝	156.8	1.14	249	3110	3950	0.79
1:5.18	.50	4¼	151.8	1.63	307	3850	4760	0.81
1:4.73	.50	7⅝	151.4	1.74	327	3970	4690	0.85
1:7.60	.51	¾	157.3	1.24	238	3780	4840	0.78
						Average		0.86

Note: Tests with Grand Coulee Dam aggregate, 1½ inch maximum size.

TABLE 13—SUMMARY OF DATA SHOWING EFFECT OF SIZE OF TEST CYLINDER ON THE COMPRESSIVE STRENGTH OF CONCRETE

Reference	Relative Strengths in Various Size Cylinders, Percent 2"x4"	3"x6"	6"x12"	8"x16"	12"x24"	18"x36"	24"x48"	36"x72"
% of 6"x12" values, Table 5 and Fig. 4, Original analysis	108	106	100	96	92	86	84	84
% of 6"x12" values, Table 11, New analysis	107	104	100	94	92	88	88	92
% of 6"x12" values, Table 12, Grand Coulee tests	—	—	100	—	—	—	86	—
% of 6"x12" values, Figure 22, Gilkey's discussion	107	104	100	95	93	86	87	87

EFFECT OF CEMENT CONTENT AND SAND-GRAVEL RATIO

As previously mentioned, the original tests with Boulder Dam aggregate for both the effect of size of aggregate and wet-screening involved variations in cement content and sand-gravel ratios for a given water-cement ratio and consistency. Tests with Grand Coulee Dam aggregates and the same cement as used in the Boulder Dam tests have now been completed and indicate that within the range of variation normally encountered in wet-screening or varying size of

[1] "Mass Concrete for Boulder Dam—Its Development and Characteristics" by Byram W. Steele, *Engineering News-Record*, vol. III, no. 25, p. 739, Dec. 21, 1933.

TABLE 14—EFFECT OF CEMENT CONTENT ON COMPRESSIVE STRENGTH AND OTHER PROPERTIES OF CONCRETE

Cement to Aggregate by Wt.	Sand to Gravel by Wt.	W/C by Wt.	Max. Aggreg. In.	Slump In.	Unit Wt. Lbs. per Cu. Ft.	Cement Content Bbl./c.y.	Water Content Lb./c.y.	28-Day Str., p.s.i.	
								24"x48" Cyls.	6"x12" Cyls.
1:6.75 1:5.86	1:1.25 1:1.25	.66 .66	1½ 1½	7½ 10	150.6 150.3	1.29 1.44	320 358	2440 2680	2900 2510
1:5.86 1:5.18	1:1.25 1:1.25	.57 .57	1½ 1½	6⅛ 10	151.7 150.8	1.47 1.60	315 343	3240 3060	3660 3300
1:5.18 1:4.73	1:1.25 1:1.25	.50 .50	1½ 1½	4¼ 7⅝	151.8 151.4	1.63 1.74	307 327	3850 3970	4760 4690
1:10.82 1:10.50 1:9.40	1:2.61 1:2.61 1:2.61	.67 .67 .67	6 6 6	2¼ 4⅝ 7⅝	157.3 157.3 157.4	0.90 0.93 1.02	228 234 257	2460 2410 2520	— — —
1:9.40 1:8.86 1:8.32	1:2.61 1:2.61 1:2.61	.58 .58 .58	6 6 6	2¼ 4⅝ 7½	158.6 158.0 158.6	1.03 1.09 1.15	225 238 251	3040 3160 3110	— — —
1:8.32 1:7.96 1:7.60	1:2.61 1:2.61 1:2.61	.51 .51 .51	6 6 6	2¼ 3½ 5⅜	158.8 158.4 158.6	1.16 1.20 1.25	223 230 240	3720 3840 3810	— — —

Note: Tests made with Grand Coulee Dam aggregate.

TABLE 15—EFFECT OF SAND-GRAVEL RATIO AND CEMENT CONTENT ON COMPRESSIVE STRENGTH AND OTHER PROPERTIES OF CONCRETE

Cement to Aggregate by Wt.	Sand to Gravel By Wt.	W/C By Wt.	Max. Aggreg. In.	Slump In.	Unit Wt. Lbs. per Cu. Ft.	Cement Content Bbl./c.y.	Water Content Lb./c.y.	28-Day Str., p.s.i. 24"x48" Cyls.
1:6.75 1:9.40	1:1.25 1:2.61	.66 .67	1½ 1½	7½ 2⅜	150.6 156.8	1.29 1.02	320 257	2440 2610
1:6.75 1:9.40	1:1.25 1:2.61	.66 .67	6 6	7¾ 7½	152.8 157.4	1.30 1.02	323 257	2390 2520
1:5.86 1:8.32	1:1.25 1:2.61	.57 .58	1½ 1½	6⅛ 1⅝	151.7 156.8	1.47 1.14	315 249	3240 3110
1:5.86 1:8.32	1:1.25 1:2.61	.57 .58	6 6	7⅜ 7¼	152.4 158.6	1.47 1.15	315 251	3110 3110
1:5.18 1:7.60	1:1.25 1:2.61	.50 .51	1½ 1½	4¼ ¾	151.8 157.3	1.63 1.24	307 238	3850 3780
1:5.18 1:7.60	1:1.25 1:2.61	.50 .51	6 6	7¼ 5⅜	153.5 158.6	1.65 1.25	310 240	3820 3810

Note: Tests with Grand Coulee Dam aggregate.

aggregate that neither cement content nor sand-gravel ratio materially affect the compressive strength. The results of the Grand Coulee tests are shown in Tables 14 and 15.

The data presented in Tables 14 and 15 indicate there are no compensating effects on compressive strength due to varying cement content or sand-gravel ratio in concretes containing various maximum sizes of aggregate or in wet-screened concrete, as compared with the corresponding full mix, provided the water-cement ratio remains unchanged.

EFFECT OF MAXIMUM SIZE OF AGGREGATE

The analysis of the data from Table 5 showing the effect of maximum size of aggregate is given in Table 16. The direct experimental factors shown in Table 11 were used for converting the test values listed in

Table 16 to a comparable basis, i.e., equivalent strengths in 24- by 48-in. cylinders. No consistent relationship between maximum size of aggregate and compressive strength is found except that the ¾-in. maximum size mixes gave uniformly higher strengths than the other sizes, and the conclusion is drawn that maximum size of aggregate has no appreciable effect upon compressive strength for the conditions under which these tests were made.

The above conclusion is substantiated by the results of additional tests made with Grand Coulee Dam aggregate as shown in Table 17. The data presented in Tables 14 to 17 inclusive indicate that variations in compressive strength as a result of varying maximum size of aggregate for a given water-cement ratio are of no practical significance regardless of accompanying variations in cement content, sand-gravel ratio or consistency. In other words, the only factor indicated by these tests as appreciably affecting compressive strength is water-cement ratio. It should be borne in mind that the sand grading and cement-sand ratio for a given set of tests were maintained constant.

EFFECT OF WET-SCREENING

From the preceding discussion, the inference is logically drawn that wet-screening should not affect the compressive strength as compared with the full mix provided both the wet-screened and full-

TABLE 16—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON COMPRESSIVE STRENGTH AND OTHER PROPERTIES OF CONCRETE

Mix by Wt.	W/C by Wt.	Max. Aggreg. In.	Slump In.	Unit Wt. Lbs. per Cu. Ft.	Cement Content Bbl./c.y.	Water Content Lb./c.y.	Avg. 28 and 90-Day Strength		
							Size of Test Cyls.	p.s.i.	Equiv. Str. in 24"x48" Cyls.
1:3.06:8.81	.66	9	3	155.0	0.82	203	36"x72"	2860	2730
1:3.06:8.08	.66	6	2¾	154.5	0.87	216	24"x48"	2590	2590
1:3.06:6.62	.66	3	3½	152.8	0.97	241	12"x24"	2840	2710
1:3.06:5.43	.66	1½	2	153.8	1.09	270	6"x12"	2860	2440
1:3.06:4.50	.66	¾	2	150.3	1.17	290	6"x12"	3280	2800
1:2.45:7.05	.54	9	3	155.6	1.01	205	36"x72"	3380	3230
1:2.45:6.47	.54	6	3¼	155.9	1.07	217	24"x48"	3280	3280
1:2.45:5.30	.54	3	3¼	153.9	1.19	242	12"x24"	3530	3360
1:2.45:4.35	.54	1½	3	154.8	1.33	270	6"x12"	3810	3260
1:2.45:3.60	.54	¾	3¾	152.3	1.44	292	6"x12"	4240	3620
1:2.04:5.88	.47	9	3½	156.4	1.20	212	36"x72"	4120	3940
1:2.04:5.39	.47	6	3½	155.4	1.25	221	24"x48"	3680	3680
1:2.04:4.41	.47	3	3¾	153.9	1.40	247	12"x24"	4180	3980
1:2.04:3.62	.47	1½	2¾	152.7	1.54	272	6"x12"	4280	3660
1:2.04:3.00	.47	¾	3	150.3	1.66	293	6"x12"	4970	4250
Average for each maximum size of aggregate									
1:2.52:7.24	.56	9	3⅛	155.7	1.01	207	36"x72"	3450	3290
1:2.52:6.64	.56	6	3⅛	155.3	1.06	218	24"x48"	3180	3180
1:2.52:5.44	.56	3	3½	153.5	1.19	243	12"x24"	3520	3350
1:2.52:4.46	.56	1½	2⅝	153.8	1.32	271	6"x12"	3650	3120
1:2.52:3.70	.56	¾	2¾	151.0	1.42	292	6"x12"	4160	3560

Note: Tests made with Boulder Dam aggregate.

TABLE 17—EFFECT OF MAXIMUM SIZE OF AGGREGATE ON COMPRESSIVE STRENGTH AND OTHER PROPERTIES OF CONCRETE

Cement to Aggregate By Wt.	Sand to Gravel By Wt.	W/C By Wt.	Max. Aggreg. In.	Slump In.	Unit Wt. Lbs. per Cu. Ft.	Cement Content Bbl./c.y.	Water Content Lb./c.y.	28-Day Str., p.s.i., in 24"x48" Cyls.
1:6.75 1:6.75	1:1.25 1:1.25	.66 .66	1½ 6	7½ 7¾	150.6 152.8	1.29 1.30	320 323	2440 2350
1:9.40 1:9.40	1:2.61 1:2.61	.67 .67	1½ 6	2⅜ 7½	156.4 157.4	1.02 1.02	257 257	2610 2520
1:5.86 1:5.86	1:1.25 1:1.25	.57 .57	1½ 6	6⅛ 7⅜	151.7 152.4	1.47 1.47	315 315	3240 3110
1:8.32 1:8.32	1:2.61 1:2.61	.58 .58	1½ 6	1⅝ 7¼	156.8 158.6	1.14 1.15	249 251	3110 3110
1:5.18 1:5.18	1:1.25 1:1.25	.50 .50	1½ 6	4¼ 7¼	151.8 153.5	1.63 1.65	307 310	3850 3820
1:7.60 1:7.60	1:2.61 1:2.61	.51 .51	1½ 6	¾ 5⅜	157.3 158.6	1.24 1.25	238 240	3780 3810
Average of All Tests			1½ 6	3¾ 7	154.1 155.6	1.30 1.31	281 283	3170 3120

Note: Tests with Grand Coulee Dam aggregate.

mix concretes are tested in 18- by 36-in. or larger cylinders. Furthermore, the relationship between the strength of full mix concrete containing large aggregate tested in large cylinders and corresponding wet-screened concrete tested in small cylinders should merely show the effect of cylinder size. However, the data obtained consistently indicate higher strengths for wet-screened concrete than for the full mix.

The analysis of the data from tests of wet-screened concretes shown in Table 5 is given in Table 18. As indicated by the average relative values, wet-screening the 6- and 9-in. maximum-size mixes to 1½ in. increased the strength of the concrete about 12 per cent. However,

TABLE 18—EFFECT OF WET-SCREENING ON THE COMPRESSIVE STRENGTH OF CONCRETE

Full Mix					Full Mix Wet-Screened to 1½ In. Max. Size					
								Equivalent Strengths[1]		
Cement to Agg. by Wt.	W/C by Wt.	Max. Size Agg. In.	Cyl. Size In.	28-D Unit Str. P.s.i.	Cyl. Size In.	28-D Unit Str. P.s.i.	% of Full Mix Str.	Cyl. Size In.	P.s.i.	% of Full Mix Str.
1:11.87 1:11.14 1: 9.68	0.66 0.66 0.66	9 6 3	36x72 24x48 12x24	2600 2290 2560	6x12 6x12 6x12	3290 3120 2790	127 136 109	36x72 24x48 12x24	2740 2640 2540	105 115 99
1: 9.50 1: 8.92 1: 7.75	0.54 0.54 0.54	9 6 3	36x72 24x48 12x24	3090 3020 3180	6x12 6x12 6x12	4050 4060 3520	131 134 111	36x72 24x48 12x24	3380 3420 3200	109 113 100
1: 7.92 1: 7.43 1: 6.45	0.47 0.47 0.47	9 6 3	36x72 24x48 12x24	3740 3370 3860	6x12 6x12 6x12	4610 5040 3920	123 150 102	36x72 24x48 12x24	3840 4250 3570	103 126 92
	Average for 6" and 9" max. size mixes						134			112

Note: Tests made with Boulder Dam aggregate.
[1]The strengths obtained in 6- by 12- in. cylinders were corrected for effect of size of cylinder by the factors shown in Tabel 11.

wet-screening the 3-in. maximum-size mix produced no change in the strengths. Similarly, tests made with Grand Coulee Dam aggregate, reported in Table 19, show an increase in strength of 9 per cent after wet-screening the 6-in. maximum-size mixes to 1½ in. maximum size.

TABLE 19—EFFECT OF WET-SCREENING ON THE COMPRESSIVE STRENGTH OF CONCRETE

Full Mix. 6 Inch Max. Size				Full Mix Wet-Screened to 1½ In. Max. Size					
Cement to Agg. by Wt.	W/C by Wt.	Cyl. Size In.	28-D Unit Str. P.s.i.	Cyl. Size In.	28-D Unit Str. P.s.i.	% of Full Mix Str.	Cyl. Size In.	28-D Unit Str. P.s.i.	% of Full Mix Str.
1:10.82	0.67	24x48	2460	24x48	2620	106	6x12	3110	126
1:10.50	0.67	24x48	2410	24x48	—	—	6x12	3200	133
1:9.40	0.67	24x48	2520	24x48	2680	106	6x12	2900	115
1:9.40	0.58	24x48	3040	24x48	3620	119	6x12	4090	134
1:8.86	0.58	24x48	3160	24x48	—	—	6x12	3940	125
1:8.32	0.58	24x48	3110	24x48	3340	107	6x12	3600	116
1:8.32	0.51	24x48	3720	24x48	3850	104	6x12	4760	128
1:7.96	0.51	24x48	3840	24x48	—	—	6x12	4820	126
1:7.60	0.51	24x48	3810	24x48	4260	112	6x12	5030	132
			Average			109			124

Note: Tests made with Grand Coulee Dam aggregate.

Tests made with Boulder Dam aggregate to show the effect of wet-screening the 9-in. maximum-size mix to other maximum sizes were made with mixes proportioned equivalent to wet-screening and mixed without the cobbles. Strengths and relative strengths of mixes so proportioned are given in in Table 20, and similar values from tests with Grand Coulee aggregate for mixes actually wet-screened are given in Table 21.

TABLE 20—STRENGTHS OF MIXES PROPORTIONED EQUIVALENT TO WET-SCREENING

Mix	Cement to Agg. by Wt.	Sand to Gravel by Wt.	W/C by Wt.	Max. Aggreg. In.	Average 28 and 90-Day Strengths			
					24x48 In. Cyl.		18x36 In. Cyl.	
					P.s.i.	Relative	P.s.i.	Relative
Full	1:9.5	1:2.88	0.54	9	3120	95.1	3160	95.2
Proportioned Equivalent To Wet-screening	1:8.86	1:2.62	0.54	6	3280	100.0	3320	100.0
	1:7.39	1:2.00	0.54	3	3580	109.1	3620	109.0
	1:5.74	1:1.35	0.53	1½	3700	112.8	3580	107.8
	1:4.39	1:0.75	0.55	¾	3690	112.5	3720	112.1

Note: Tests made with Boulder Dam aggregate.

TABLE 21—STRENGTHS OF MIXES ACTUALLY WET-SCREENED

Mix	Cement to Agg. by Wt.	Sand to Gravel by Wt.	W/C by Wt.	Max. Size Aggreg. In.	28-Day Unit Strengths 18x36 In. Cyl.	
					P.s.i.	Relative
Full	1:9.40	1:2.61	0.58	6	3020	100.0
Actually Wet-screened	1:7.70	1:1.96	0.58	3	3220	107.0
	1:5.87	1:1.26	0.58	1½	3380	112.0
	1:4.37	1:0.68	0.58	¾	3490	116.0

Note: Tests made with Grand Coulee Dam aggregate.

A summary of the data presented fixes the relative strengths of full mix and corresponding wet-screened concretes fabricated with natural aggregates and cured under standard conditions as follows:

6- or 9-in. maximum-size concrete tested in 18- by 36-, 24- by 48-, or 36- by 72-in. cylinders	77 per cent
6- or 9-in. maximum-size concrete wet-screened to 1½-in. maximum size and tested in 18- by 36-, 24- by 48-, or 36- by 72-in. cylinders	84 per cent
and in 6- by 12-in. cylinders	100 per cent

In other words, wet-screening concrete containing 6- or 9-in. maximum-size aggregate to 1½ in. maximum size increases the strength some 9 per cent, which, from other data presented, cannot be explained on the basis of cement content, size of aggregate or sand-gravel ratio. It is possible that the increase resulting with wet-screening is a combination effect of several factors, any one of which is too small to detect singly.

Data from other tests show that mass curing conditions increase the full-mix concrete (fabricated with an average cement) strengths about 10 per cent at 28 days. Stated in the terms listed above, this means that the full-mix concrete containing 6- or 9-in. aggregate in 18- by 36-in. or larger cylinders, cured under mass concrete conditions and tested at 28 days will yield about 85 per cent of the strength of the same concrete after wet-screening to 1½-in. maximum size and tested in 6- by 12-in. cylinders cured under standard conditions.

CONCLUSION

The additional studies and data have not changed the conclusions presented in the original paper with the exception of No. 15 which states that the effect of wet-screening on compressive strength is of no appreciable importance. The effect is small but consistent and must therefore be recognized.

It is regretted that space does not permit detailed discussion of the individual points raised by Professor Gilkey but it is hoped the additional discussion, data and analysis presented herein will serve to clarify the major questions involved.

ACKNOWLEDGMENTS

All activities of the Bureau of Reclamation are supervised by Dr. Elwood Mead, Commissioner of Reclamation, Washington, D. C. All engineering and construction is under the general direction of R. F. Walter, Chief Engineer, with headquarters in Denver. All research and design work is under the direction of J. L. Savage, Chief Designing Engineer, and the investigations herein described were conducted under the general supervision of B. W. Steele, Designing Engineer on Dams.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Concrete Highways in Germany

German Cement Association, Berlin, 1935. Reviewed by INGE LYSE.

A comprehensive booklet on modern concrete highway construction methods in Germany. This year's issue is of particular interest because of the activity in concrete highway construction initiated by the far reaching scheme of the Hitler automobile road system for the whole country. Methods of construction including mechanical equipment are described. Many excellent pictures.

Rolled concrete pavements in Victoria, Australia

M. G. DEMPSTER, (Engineer for Bridges, Country Roads Board, Victoria, Australia), *Engineering News-Record*, Vol. 115, No. 13, Sept. 26, 1935, p. 425-426. Reviewed by N. M. NEWMARK.

Excellent results are claimed in the construction of roller compacted concrete pavements with mixes as lean as 1:2 1/2:10. Uniformly high moduli of rupture have been obtained from tests of beams cut from the completed pavements. A three mile road 30 ft. wide has been successfully constructed.

Rapid methods of analysis in the cement industry

F. W. MEIER, *Zement*, No. 34, Aug. 22, 1935, p. 517. Reviewed by INGE LYSE.

Recent years have seen the development of rapid and accurate methods for the determination of the chemical elements in cement. The author discusses methods for determining the SiO_2, Al_2O_3 and Fe_2O_3 and presents examples of the details of procedure. The paper should be of considerable interest to all who are engaged in the chemical analyses of cement, both in the manufacturing and the research fields.

English specifications for blast furnace cement

Zement, No. 39, page 609, Sept. 26, 1935. Reviewed by INGE LYSE.

Specifications deal with cement made from a mixture of portland cement clinker and granulated blast furnace slag—the slag content limited to a maximum of 65 per cent. The two materials must be ground together to achieve thorough mixing. The cement must pass the requirements as to fineness, chemical composition, tensile strength (300 and 375 p.s.i. at three and seven days respectively) of mortar, time of set, and volume change.

Monument to General Obregon

ENRIQUE ECHEAGARAY, Architect. Pamphlet issued by author Reviewed by C. G. and M. N. CLAIR.

A collection of photographs with comments by prominent engineers, architects and sculptors on the monument at Obregon, Mexico to General Obregon. The

principal feature of the structure is a massive octagonal hollow shaft of rough form-finish concrete. The beauty and solemnity created by the simple exterior and interior use of concrete warrants the attention of all interested in architectural concrete.

Relation between chemical composition, manufacture, and physical properties of portland cements

D. STEINER, *Zement*, No. 31 and 32, Aug. 1 and 8, 1935. Reviewed by INGE LYSE.

This paper presents the results of the first part of an extensive study of the properties of cements of various compositions, a large number of which were produced for this study. The chemical analyses as well as the strength results are given for 16 different cements. The results are given both in tabular and graphical forms and discussed with respect to the different variables.

Electro concrete

ANDREAS RETHY, *Teknisk Tidskrift*, Sept. 28, 1935. Reviewed by INGE LYSE.

About four years ago two Swedish engineers, A. Brund and H. Bohlin published a proposal for heating concrete in cold weather by means of electric current. This method has been accepted by Soviet Russia as promising winter construction procedure and the production schedule for this winter is three times as great as for last winter. The method has been studied thoroughly whereby scientific basis of production has resulted. This article, in Swedish, describes the details of procedure and shows illustrations of arrangements for construction and of final concrete products.

Expanded metal for concrete highways

WALTER HARTLEB and ALFRED BERRER, *Die Betonstrasse*, No. 9, Sept. 1935, p. 175. Reviewed by INGE LYSE.

The increased interest in concrete roads in Germany has created a demand for more information on the comparative merits of different types of shrinkage reinforcement used. The investigation discussed in this article was made at the Technical Institute of Breslau. A detailed description is given of the materials and methods of construction of specimens as well as a tabulation of the test results. The authors conclude that on the basis of elongation at first appearance of crack the expanded metal was superior to the ordinary reinforcing steel.

Calcium-ferro-hydrate; a contribution to the hardening theory for portland cement

HELLMUTH HOFFMANN, Bull. 52, Cement Institute of Technical Institute of Berlin, 1935. Reviewed by INGE LYSE.

Prof. Hans Kühl has written a foreword as an introduction for this bulletin which contains a report on a thorough study of one of the cement compounds. The report gives an excellent review of previous work in the study of calcium-ferro-hydrate and presents an extensive bibliography. The experimental work of the author is described in detail, both with respect to materials and test methods, and the results are summarized in ten conclusions. The bulletin contains 69 pages.

Grand Coulee Dam

Engineering News-Record, Vol. 115, No. 5, Aug. 1, 1935, pp. 141-160. Reviewed by N. M. NEWMARK.

Seven articles describing the Grand Coulee Dam under construction on the Columbia River in northern Washington are continued in this issue. The completed dam will be three times as big as Boulder Dam, its main function being to provide irrigation water. At present only the base of the future dam is being built.

The articles give a survey of the entire project and describe construction progress to date, the building of the first cofferdam, the plans for making and handling the aggregate, and the manner of spoils disposal by means of belt conveyors.

Plastic Mortar for cement testing

G. Haegermann, *Zement*, No. 35, Aug. 29, 1935, p. 529. Reviewed by Inge Lyse.

A discussion of the different methods for testing the quality of cement for concrete. Concrete highway construction has brought forth a demand for test of flexural and tensile strength as well as compressive strength. The author shows that the relation between tensile and compressive strengths of concrete follows approximately the formula:

$$f_t = \sqrt{2.6 f_c}$$

where f_t and f_c represent tensile and compressive strength respectively. This formula gives the upper limit of the tensile strength for moist cured concrete.

Absorption of water by portland cement paste during the hardening process

T. C. Powers, *Industrial and Engineering Chemistry*, July, 1935, Vol. 27, No. 7, p. 790-794. Reviewed by R. N Young.

The physical and chemical changes in a mixture of cement and water result in a decrease of the total volume. At the same time, however, the dimensions of the solid phase or hardened paste may increase. This phenomenon of volume change of the water-cement system was studied with relation to chemical composition of the cement, water-cement ratio and heat of hydration. The degree of volume change is suggested as an index of heat of hydration but a definite relationship was not established.

Unusual concrete roof covers British sports building

Engineering News-Record, Vol. 115, No. 6, Aug. 8, 1935, p. 196-197. Reviewed by N. M. Newmark.

The reinforced concrete building housing a large swimming pool at Wembley near London has walls and roof consisting of a three-hinged arch of 236½ ft. span. The spectators galleries and lavatories are built integrally with the roof. The arch ribs, spaced on 22 ft. centers, are 9 in. wide and increase in depth from about 6 ft. near the crown to 21 ft. at the side walls. The side walls are 2 ft. 9 in. wide and 10 ft. 6 in. deep to a point below the gallery connection. The roof and side-wall ribs stick out from the buildings like fins. At the crown the ribs are cut away, leaving only horizontal slabs 6 in. deep and 2 ft. 9 in. wide to pass through to form a quasi hinge.

The hydraulic equilibrium of calcium-alumina-hydrate

E. P. von Polheim, Bull. 51, Cement Institute of Technical Institute of Berlin, 1935. Reviewed by Inge Lyse.

This bulletin, with foreword by the outstanding German cement scientist, Prof. Hans Kühl, reports an investigation at the Cement Institute and deals with previous work on the problem as well as with the data obtained by the author. The investigation, well planned and carefully executed, contributed much additional information on one specific cement compound and should therefore be valuable to everybody interested in the chemistry of cement and concrete. Professor Kühl's Institute has for years contributed greatly to the knowledge of cement and its hydration processes, and Mr. Polheim's report is a timely addition to the many excellent papers which have been published by the Institute.

Portland cement and its possibilities

R. W. Carlson, *Industrial and Engineering Chemistry*, Sept., 1935, Vol. 27, No. 9, pp. 1014-1016. Author's Abstract

Simple descriptions are given of the changes which take place in limestone and clay as they pass through the kiln and grinding mills to produce portland cement, and in the cement as it hydrates to produce hardened concrete. The limited demands

made upon concrete by the civil engineer indicate that the strength-giving quality of cement can be used only in small measure because of the properties of volume change and brittleness in the concrete. The major factors which influence volume change of concrete are given, and suggestions are made as to possible methods of controlling volume changes. One ot these suggestions is to increase the efficiency of the cement so that less can be used per cubic yard of concrete.

Compressive strength of concrete in flexure and under eccentric loading

ANTON BRANDTZAEG, Trondhjem, 1935. Reviewed by INGE LYSE.

This bulletin, in German, reports an investigation at the Norwegian Institute of Technology, of the stress condition at failure of reinforced concrete beams, and reinforced concrete columns subjected to eccentric loading. Twenty beams and thirteen columns were included. Detailed description is given of materials and methods of manufacture and test for the full-sized test specimens, as well as for the concrete cubes and cylinders used as control specimens. One of the principal conclusions points out the agreement between computed strength by means of parabolic stress distribution and strength of concrete as obtained on cubes. The bulletin contains 87 pages with 14 tables and 57 figures.

Tensile strength of concrete

A. GUTTMANN, *Zement*, No. 35, Aug. 29, 1935, p. 532. Reviewed by INGE LYSE.

The engineer will generally assume the tensile strength of concrete to range between one-tenth and one-twelfth of the compressive strength. To secure more data on the relationship between tensile and compressive strengths of concrete, the author made a series of tests. The tensile bars had rectangular cross section about four by four inches with a length of about three and one-half feet. The results showed a great variation in the ratio between tensile and compressive strength and the author recommends that test determination should be made wherever high tensile stresses are used in the concrete. Relatively high tensile strength may be obtained by the selection of the proper cement, the best aggregate using a high mortar content, the use of a minimum of mixing water, and the application of moist curing during the early hardening period.

Bridge over Kalix river at Tarendo

IVAR HAGGBOM, *Betong*, No. 3, 1935, p. 100-107. Reviewed by INGE LYSE.

This article describes the construction of a reinforced concrete bridge, north of the Arctic Circle in Sweden and therefore subjected to severe weather conditions. It has a span of 125 m. (410 ft.) and consists of two hollow fixed arches from which the roadway is suspended. Since the average temperature for the year was only about 20°F, high temperature stresses were taken into account in the design. These secondary stresses were found to be as much as 60 per cent of the stresses caused by live load. The arch sections were designed for maximum compressive stress of 1000 p.s.i. and minimum compressive stress of 140 p.s.i. The 28 days compressive strength of the control cubes was placed at 4600 p.s.i. Four hydraulic jacks with capacity of 100 tons each were used at the crown of each arch section to produce compression in the section before the last portion of the concrete was placed.

Flexural resistance of shallow concrete beams

CONDE B. MCCULLOUGH, (Oregon Highway Department), *Engineering News-Record*, Vol. 115, No. 12, Sept. 19, 1935, p. 406-407. Reviewed by N. M. NEWMARK.

Results are given of tests of 20 shallow reinforced concrete beams, each 24 in. wide, 10 in. in effective depth, and 12½ ft. between supports, differing in amount of longi-

tudinal reinforcement and in the manner of web reinforcement. The yield point of the steel averaged about 50,000 p.s.i. and the ultimate strength of the concrete about 4000 p.s.i.

Failure of the concrete in compression occurred only with amounts of longitudinal steel greater than 3.7 per cent. The compressive stress computed by the straight line formulas with $n = 8$ averaged about 1.7 times the cylinder strength. These results are apparently in conformity with those reported in *Proceedings*, Amer. Concrete Inst., by Slater and Zipprodt, Vol. 16, 1920, p. 120-146, and by Slater and Lyse, Vol. 26, 1930, p. 831-874, and, as has been pointed out in these papers, are to be expected as a direct consequence of the shape of the stress-strain curve for concrete.

The history of the development of the methods of design of reinforced concrete

A. Muncz, *Ingenieria*, Vol. 9, No. 1, June 1935, p. 340 Reviewed by C. G. and M. N. Clair.

First article of series on principles of design of reinforced concrete. The historical development is traced starting with F. Coignet and Lacroix in 1861 who first recognized that the addition of steel increased the load carrying capacity of concrete. Joseph Monier in 1865 received the first patent for reinforced concrete construction and this patent applied to boxes and containers for horticultural applications. Later patents by Monier covered pipe and structures and in 1880 a company was formed in Berlin to promote construction under his patents. This organization did a great deal of experimental work and developed a good basis for use of reinforced concrete. W. E. Ward built the first concrete house in America in 1872 at Port Chester, New York. The work of Koenen, E. Coignet, Klein, Hennebique, Ritter, Moersch, Melan, in the development of a rational basis for analysis of reinforced concrete is briefly reviewed. Credit is given to the Joint Committee on Concrete and Reinforced Concrete for codification of practice in the United States.

A study for the preparation of a specification for high-early-strength portland cement

G. Rupert Gause. Research Paper (RP 839), *National Bureau of Standards Journal of Research*, Vol. 15, Oct. 1935. Author's Abstract

This paper reports test results to be used as a basis for the preparation of a Federal specification for high-early-strength cement. Investigation was made of samples of 28 commerical cements representing a wide spread in compound composition, fineness, and physical properties.

Four plastic mortars were studied. Both the tensile and compressive strength (2-in. cubes) of each of these mortars varied with the different cements over a considerable range. The rate of setting was measured by the penetration of 300-g needles, one 1 mm in diameter and one 2 mm in diameter, into the mortars contained in a Vicat ring.

Measurements of length changes were made on 6-in. prisms 1 in. square cured under four different conditions.

The requirements for a specification for high-early strength cement are discussed and recommendations made for tests to be incorporated in such a specification.

Acid resistance of cement pipes

E. Suenson, Bul. No. B15, Danish Engineering Society, Copenhagen, 1935. Reviewed by Inge Lyse.

This bulletin describes in detail the extensive research studies which for many years have been conducted at the Danish Institute of Technology under the able direction of Professor Suenson. From investigations on sewer pipes as well as mortar

bars which were made, treated, and tested under different conditions, the results will be of great assistance to everyone interested in the factors which contribute to the disinetgration of concrete. Lack of space prevents the presentation of any of the many interesting findings reported in this bulletin, but since a rather complete summary is given in English, French, German, as well as in Danish, in which the report proper is presented, it is recommended that those interested read this summary. The report proper, excluding the foreign summaries, contains 230 pages with 47 figures and 33 tables. In addition, a complete bibliography of recent important papers on durability, permeability, and similar subjects is given. Professor Suenson has by this report made an outstanding contribution to the advancement of our knowledge of concrete disintegration and he and his assistants should be congratulated upon the manner in which the work was carried out and the results reported.

The effect of aggressive solutions upon hardened cement

A. STEOPOE, *Tonindustrie Zeitung*, Vol. 59, No. 64, p. 765-8, Aug. 8, 1935. Reviewed by A. E. BEITLICH.

Pulverized samples of hardened specimens of neat cemênt (which were made porous by admixtures of especially prepared calcium carbide to increase the effective surface) were treated with distilled water and solutions of magnesium sulfate (1.25 per cent) and sodium chloride (10-20 per cent). The filtered solutions and the remaining residue were analyzed.

Distilled water has a simple dissolving effect upon the calcium hydroxide of the cement. In the case of magnesium sulfate a number of reactions take place aside from the calcium oxide going into solution as calcium sulfate and the residue takes up more magnesium hydroxide and sulfur trioxide than the amounts which would correspond to the dissolved calcium hydroxide. Sodium chloride has a strong dissolving action upon the calcium hydroxide of hardened cement. It dissolves even more than a weak magnesium sulfate solution or distilled water. The effect of the carbon dioxide of the air or a preliminary treatment of the hardened cement with distilled water or sodium chloride solution reduces considerably the effect of aggressive solutions upon hardened cement.

Thermal Effects during the setting of cement

A. S. KLEIN, *Le Constructeur de Ciment Arme*, V. 17,No. 191, Aug., 1935. Reviewed by P. H. BATES

The author briefly discusses the development of heat in concrete due to the reaction of the cement and water. He then presents the usual formula for determining the time required for the concrete to reach a certain temperature based upon an observation of the rates of cooling the concrete, the thermal conduction of the concrete and its specific heat. He also discusses the work of Woods, Steinour and Stark of the Riverside Cement Co., published in the *Engineering News Record*, Oct. 6 and 13, 1932, and Apr. 6, 1933. He takes the results the authors have presented covering the relation between the heat developed at certain ages and the chemical composition and presents two g aphs based upon the relation between the heat developed and the percentage of iron oxide, the ratio of silica plus alumina plus iron oxide to lime plus magnesia and the ratio of alumina to silica. The graphs show that there is little relation between the per cent of iron oxide in the cement and the heat. There is a slight trend showing that with less iron oxide the heat is also less. Until the heat developed by the cement is greater than 90 calories per gram, the alumina-silica ratio is constant, but for heats greater than this the ratio increases rapidly with the heat developed. As the ratio of silica plus alumina plus iron oxide to lime plus magnesia decreases there is a consistent increase in the amount of heat developed.

Australian specification for cast stone

Concrete Building and Concrete Products, Vol. X, No. 10, Oct. 1935, p. 192-4.
Reviewed by J. C. Pearson.

The Standards Association of Australia has issued a standard specification for cast stone, which is defined as a "pre-cast synthetic building stone manufactured with a portland cement base." The chief requirements are as follows:

Minimum average compressive strength at 28 days, or earlier, 5000 p.s.i.

Average absorption at 28 days, not more than 7 per cent by weight.

Specimens for compression and absorption tests shall be 2 in. x 2-in. cylinders, or 2-in. cubes, but so that the test load is applied in a direction perpendicular to the bed plane. Three specimens shall be tested for compression, and three for absorption, and specimens used for the latter shall not be used for the compressive test. One set of specimens for each test shall be made for each lot of 10,000 cu. ft. of cast stone or portion thereof. Failure shall be based on average values, and in case of failure in either test, six additional specimens shall be required for a retest. Acceptance or rejection shall be based on the average value obtained from the second test.

The usual procedure is prescribed for making the compression test. The specimens are to be oven-dry at 215-225° F. when the test is made, and the load application rate shall be 5000 p.s.i. per minute.

The absorption test is to be made on oven-dry specimens submerged in water between 60 and 80° F. for 24 hours.

Plastic flow of portland cement concrete

J. R. Shank, Industrial and Engineering Chemistry, Sept., 1935, Vol. 27, No. 9, pp. 1011-1014.
Reviewed by R. N. Young.

This paper is a study of test data available from investigations at the University of California, The Bureau of Building Research in England, and the Ohio State University. The phenomenon is discussed and the following equation is proposed to indicate the plastic flow property of concrete:

$$y = C\sqrt[a]{x}$$

where y = plastic deformation, inches/inch/pound/square inch
x = time, days
C = coefficient drawn from tests
a = a root drawn from tests

The average value for C computed from the data was 0.130 and for a, 2.9. The computed values for y conformed with the observed data fairly well up to the period of 1 year, but were too high at later periods. It appears from the data studied that the coefficient C for ordinary concrete varies with age according to the following equation:

$$C = \frac{0.493}{A^{1/2.5}}$$

where A = age in days at loading, or the time in days from the date of pouring to the date when the load is applied.

Stresses in the steel reinforcement of reinforced concrete structures

R. H. Evans, *Structural Engineer* (England), V. XIII (New [Series), No. 9, Sept. 1935, pp. 354-369.
Reviewed by V. P. Jensen.

Consideration of time yield of concrete leads to conclusion that secant modulus should be used to determine modular ratio n. Beam tests show that for loads below working values the rate of increase of steel stress agrees with that calculated so as to include resistance of the concrete in tension. The rate changes gradually until, at the highest loads, it is about the same as that calculated excluding tension. The

steel stress in the region of cracks, as well as the influence of time on the steel stress, decreases with increase in the percentage of web reinforcement.

The rate of deflection, as well as the total deflection, exceeds that calculated excluded tension. Bent up bars were found effective in reducing deflection. Contact stresses as load points and reactions change the shear distributions appreciably within the beam. Observed stresses in vertical and diagonal stirrups over 1-inch gage lengths rarely exceeded 60 to 70 per cent of those calculated. When vertical stirrups were used in conjunction with bars bent up at 45° it was found that the vertical stirrups had little effect until the higher loads were reached.

Tests of reinforced concrete floor slabs in service showed steel stresses and deflection to be between those calculated including and excluding tension in the concrete. The effect of a uniform floor load was estimated by means of a 9-point loading method.

The possibilities of puzzolanas in mortars and concretes

EDW. W. SCRIPTURE, JR., (Director, Master Builders Research Laboratories, Cleveland), *Engineering News-Record*, Vol. 115, No. 17, Oct. 24, 1935, p. 563-567. Reviewed by N. M. NEWMARK.

The use of puzzolanic admixtures in a number of recent structures, notably in the foundations of the Golden Gate and San Francisco-Oakland bridges, commands attention and requires consideration. Puzzolanas are material of natural, treated, or artificial origin, possessing constituents which combine with lime at ordinary temperatures in the presence of moisture to form insoluble cementitious compounds. They are used as an addition to portland cement to react with the free lime originally in the cement and that produced by hydration. Their use reduces the solubility of cement, and thereby the susceptibility to corrosion and weathering. In certain cases workability may be maintained at a reduced water-cement ratio.

Puzzolanic material of different degrees of activity may be obtained for various purposes for use in connection with cements of different properties. It is necessary to fit the puzzolana and the cement to the specific purpose. In massive structures the reduction of heat evolution can be secured with a low-heat cement and a reactive puzzolana or with a puzzolan blend. In structures exposed to sea water, resistance to corrosion is attained by use of puzzolana with low-calcium and low-aluminate cement. In ordinary building construction the puzzolana must be such that its combination with lime takes place within a short time.

Dr. Scripture's excellent discussion of what puzzolanas are and the possibilities of their use is very timely and thought provoking.

Behavior of high-early-strength cement concretes and mortars under various temperature and humidity conditions

LOUIS SCHUMAN and EDWARD A. PISAPIA. Research Paper (RP 799), *National Bureau of Standards Journal of Research*, Vol. 14, June 1935. AUTHOR'S ABSTRACT

Data were obtained on the properties of 12 high-early-strength cements, and on various mortars and concretes made from them.

All of the cements gave early strengths higher than those of ordinary portland cements. The strength of 1:2:4 concrete with C/W ratio of 1.50 by weight varied from 560 to 1,120 p.s.i. at 1 day, and from 1,590 to 2,590 p.s.i. at 3 days. Concrete stored during the first 24 hours at 90 and 110° F. was greater in strength than concrete stored at 70° F. Damp-stored specimens gave the highest strengths after 28 days. Concrete subjected to 300 alternations of freezing and thawing had slightly ower strengths than concrete stored 1 year in the damp room. Freezing and thawing

combined with drying and soaking caused severe spalling on mortars and concretes made from some of the cements.

Certain mortar specimens made with the same C/W ratio as concrete cylinders were about equal in strength to the cylinders at early ages.

The heat evolved by the cements was computed from the rise in temperature of concrete in an adiabatic calorimeter. At 90 days the heat evolved varied from 104 to 130 calories per gram of cement.

No definite relation was found between cement compound composition and strength, length changes, or resistance to freezing, thawing, drying, and soaking of mortars and concretes.

Bond resistance of high strength and vibrated concrete.

W. Fisher Cassie, *The Structural Engineer* (England), Vol. XIII (New Series), No. 8, Aug. 1935, p. 322-336. Reviewed by V. P. Jensen.

This is an account of an investigation by the author in the Dept. of Theo. and Applied Mechanics, Univ. of Illinois, under the direction of Prof. F. E. Richart. A series of tests involving 64 pull-out specimens and 16 control cylinders was to determine the effect on bond resistance of increasing the compressive strength. Another series involving 16 beams and 32 control cylinders was run to discover the effect on bond resistance of vibrating concrete immediately after pouring. It is concluded that:

"Vibration of concrete immediately after pouring," . . when "carried to the limit of consolidation, gives great compactness, freedom from voids and a good surface."

For the usual mixes vibration may be expected to increase the strength from 10 to 15 per cent.

"Vibration may be expected to increase bond resistance in both pull-out specimens and in beams by 10 to 20 per cent, and if deformed bars are used this increase in bond resistance may be three times as great.

"In beams at working loads there are relatively high bond stresses acting over short lengths of the reinforcement. For weaker concretes, this stress may be twice the theoretical bond stress, and may approximate to the value of the maximum bond resistance shown by pull-out specimens."

The advantages of vibration are yet to be reflected in specifications.

Large concrete pressure pipes

N. D. Whitman, *Civil Engineering*, Vol. 5, No. 9, Sept. 1935, p. 553. Reviewed by J. R. Shank.

Concrete pressure pipe for siphons are being designed and constructed at Los Angeles in sizes up to 12 ft. diameter by 12 ft. long, weighing 40 tons. The special features of these pipe are welded sheet metal (No. 8 to 12 U. S. gage) cylindrical membranes having welded to the ends very precisely rounded heavier gage metal rings. Outside the spigot ring is another ring of special cross-section so shaped as to facilitate the placement of lead caulking. The membranes are placed at the insides of the pipe walls against the elliptical reinforcement. The concrete at the bell end is so shaped that caulking tools may be used. Not more than $\frac{1}{16}$ in. is allowed between the spigot ring and the bell ring, and the finished inside of the pipe may not vary by more than $\frac{1}{32}$ in. Openings in the line for manholes, blow-offs, and air valves are of cast steel welded to the reinforcement assemblies and to the steel cylindrical membrane.

When a section of pipe is brought into position for erection the lead gasket is lightly caulked and the joint outside the rings is filled with mortar. A lean concrete

cradle is then placed under the pipe which is allowed to harden after which the backfill is placed. Later, when convenient, the lead gasket is given its final caulking, using an 8 pound sledge and the inside recess at the joint is filled with mortar and finished smooth and flush with the inside of the pipe.

A line test is made by filling with water at moderate pressure for a sufficient time to have the concrete take its full moisture expansion, after which the test pressure is applied from 2 to 48 hours. The leakage is seldom more than 75 gal. per inch of diameter per mile per 24 hours. Coefficients of retardation, C_s (Scobey) = 0.370 or C_w (Hazen-Williams) = 140, are easily attained.

Limitations and applications of structural analysis

HARDY CROSS, (Prof. of Structural Engineering, Univ. of Ill.), *Engineering News-Record*, Vol. 115, No. 16, Oct. 17, 1935, pp. 535-537; and No. 17, Oct. 24, 1935, pp. 571-573.
Reviewed by N. M. NEWMARK.

This paper might very well be termed a philosophy of structural analysis, particularly of analysis for use in structural design. Essentially the article concerns the desirable characteristics of analytical procedures, the fundamental concepts of structural analysis, and the interpretation of the results of analysis.

The author suggests that a method of analysis to be used in design should fulfill certain requirements, namely: It should be easy to learn and easy to remember; it should be based on familiar concepts and depend on familiar terminology; it should furnish an approximate answer at once, and revise this approximation in about the same manner that the designer chooses certain tentative proportions and then revises these proportions; and it should be flexible, meaning that it should be able to take into account variations in physical properties of the material, so that among other things, the designer can investigate the action of the structure up to failure.

Fundamentally, the stress distribution in a structure must be in equilibrium; the strains produced by the stresses must satisfy continuity; and the stresses and strains must be in the proper relationship for the material and the conditions obtaining.

But the most important problem is the interpretation of the analysis. The scheme of classification of indeterminate structures presented by Cross (in JOURNAL, Amer. Concrete Inst., March-April, 1935, *Proceedings*, Vol. 31, p. 358-368, and in *Proceedings*, Am. Soc. C. E., Oct., 1935, p. 1119-1131) was developed for this purpose. In the present paper the classification is very briefly reviewed, and the bearing on design is discussed. The significance of overstress and of chance elements are touched upon. Professor Cross presents a point of view and a philosophy that are fundamental and extremely practical.

Effect of time yield in concrete upon deformation stresses in a reinforced concrete arch bridge

WILBUR M. WILSON and RALPH W. KLUGE, Bul. 275, Engineering Experiment Station, Univ. of Ill.
Reviewed by M. R. RIDDELL.

The investigation was part of a research on reinforced concrete arches conducted by the Experiment Station in coöperation with the United States Bureau of Public Roads.

The length of a reinforced concrete arch rib changes (a) because of the immediate strain accompanying the application of a load, usually called "rib shortening," (b) because of the time yield in the concrete, (c) b cause of temperature changes, and (d) because of the shrinkage in the concrete that takes place as the concrete dries.

All changes in the length of the rib of an arch with fixed abutments change the shape of the rib, thereby producing flexural stresses similar to the flexural stresses

that are produced by moving the abutments without changing the length of the rib. Since the stress is due to a change in shape and not due to a load, it is a *deformation stress*, and, since the changes, except rib shortening, take place slowly, it would appear possible that the time-yield property of the concrete might enable the rib to assume its new shape without incurring the stresses that would be produced if the same changes in length occurred quickly.

In view of this possibility, test were made to determine the changes in reactions at th springings of an arch due to the shrinkage of the concrete, to the combined effect of shrinkage and time yield, to dead load, and to changing the span by an amount equivalent to a change in temperature of 100 deg. F. The change in span was made in five equal increments, each increment being equivalent to a change of 20 deg. F., and the changes were made at intervals of approximately 30 days. Readings were taken just before and after each change in span to determine the immediate change in the reactions. Subsequent readings were taken at intervals of approximately one week to determine the changes in reactions due to time yield.

The determination of the free lime in portland cement

Anton Hanslitschek, *Tonindustrie Zeitung*, Vol. 59, No. 46, p. 556-8, June 6, 1935.
Reviewed by A. E. Beitlich.

After having presented a detailed review of the literature on this subject (*Tonindustrie Zeitung*, Vol. 59, No. 9 and 11, p. 110-1, 144-6, Jan. 28, and Feb. 4, 1935), the author reports on his own critical studies of the proposed methods of free lime determinations. The method by Rathke, which uses water-free glycerol and absolute alcohol as solvents and subsequent titration with 0.1 *N* tartaric acid is not suitable for scientific tests but may be sufficiently accurate for plant control work. The temperature given for complete solution of the sample is too low and the endpoint of the titration is not clearly visible.

Better results were obtained with the method by Jander and Hoffmann who speed up the solution by rubbing both the sample and the glycerol with a pestle and application of heat. The titration is carried out in aqueous solution of the filtrate with 0.1 *N* hydrochloric acid. The author slightly modified the method and obtained excellent results. However, it is too slow for plant practice since the filtration of the glycerol takes from 7 to 8 hours. The phenol method worked out by Konarzewski and Lukaszewicz and by Jander and Hoffmann did not show any advantages over the alcohol-glycerol method.

In connection with this investigation, the author observed that a cement, the free lime content of which had been determined, was perfectly sound while the clinker from the same burn, ground with 3 per cent gypsum in a small laboratory mill, did not pass the soundness test. The author explains this different behavior by a partial hydration of the free lime in the cement to the harmless calcium hydroxide due to the effect of atmospheric moisture, water additions and water driven off from the gypsum at the relatively high temperatures of the grinding process. In the clinker which was protected from the effect of moisture the free lime remained in its dangerous form as calcium oxide.

Laboratory tests of three-span reinforced concrete arch bridges with decks on slender piers

Wilbur M. Wilson and Ralph W. Kluge, Bul. 270, Engineering Experiment Station, Univ. of Ill.
Reviewed by M. R. Riddell.

Bulletin 270 (see also review Bul. 269) covers investigation consisting in tests of a three-span arch series on high piers, each span being composed of a rib with spandrel columns and a deck. For one structure the deck was a considerable distance

above the rib at the crown of the arch; for a second structure the deck was so low to be integral with the rib at the crown. Each structure as originally built had no expansion joints in the deck, except over the piers. After each structure had been tested expansion joints were cut in the deck near the one-third point of each span, and the resulting structure was again tested. The structure with a high deck, both with and without intermediate expansion joints, was tested at pier heights of 20 ft., 15 ft., and 10 ft., respectively; the structure with a low deck was tested at a pier height of 20 ft. only.

Among objects was to obtain the following information relative to the structures described:

(a) The magnitude and position of the thrust due to the design load

(b) The maximum carrying capacity of the three-span structure

(c) The effect of intermediate expansion joints in the deck on the load-carrying capacity of the structure

Among the results obtained were the following:

(a) There was considerable evidence that the dead-load stress in a multiple-span structure may exceed the corresponding stress in a similar span having fixed ends.

(b) The effect of the deck was to reduce the moment at the springing due to the load, where it is all resisted by the rib, and to increase the moment over the middle of the span, where the deck acts with the rib.

(c) A deck without intermediate expansion joints increased the stiffness and the moment-resisting capacity of the central part of the structure; intermediate expansion joints reduced both of these effects.

Laboratory tests of three-span reinforced concrete arch ribs on slender piers.

WILBUR M. WILSON and RALPH W. KLUGE, Bul. 269, Engineering Experiment Station, Univ. of Ill.
Reviewed by M. R. RIDDELL.

Bulletin No. 269 is the first of two dealing with an investigation on multiple-span reinforced concrete arch structures. Bul. 269 (see review these pages Bul. 270) contains the report of laboratory tests of a single-span arch rib, and also of a structure consisting of a three-span series of arch ribs on slender piers; a later bulletin contains the report of similar laboratory tests of three-span reinforced concrete arch structures with spandrel columns and decks on slender piers.

The objects of the tests of the single-span arch rib were to try out on a simple structure the apparatus that was being built for use in testing the more complicated three-span structure; and to determine experimentally the elastic properties of a single-span arch rib which was to be used as the basis of comparison in studying the effect of the elastic deformation of the piers upon the properties of a three-span arch series. The objects of the tests of the three-span structure consisting of a rib without deck were to determine the load-carrying capacity of the structure, and to compare the values of reactions and strains measured in the laboratory with values of the corresponding quantities obtained by the elastic theory, the three-span structure consisting of a rib without deck being one which can be analyzed by an all-algebraic process.

As already noted, these tests were followed by corresponding tests of a three-span arch series with spandrel columns and deck, a structure that cannot readily be analyzed by an all-algebraic process. A comparison of results obtained by algebraic analysis with those obtained by tests, whether the two sets of results are in complete agreement or not, is helpful in judging of the dependability of the experimental work.

The investigation included tests to determine

(1) reactions at the springings due to movement of the abutments and of the tops of the piers, frequently designated the "elastic constants" of the arch;

(2) influence ordinates for reactions at the springings by applying a unit load successively at various load points;

(3) vertical deflections of the load points due to movement of the terminals, the abutments and pier bases;

(4) reactions at the springings and strain in the concrete at sections midway between the load points due to the design load; and

(5) ultimate load-carrying capacity of the structure.

The chemistry of cement and concrete

E. M. Lea and C. H. Desch. Cloth; 6 x 9 in.; pp. 429. Photographs, diagrams, tables. Published by Edward Arnold & Co., London; and Longmans, Green & Co., New York, N. Y. $9.50.

Reviewed by P. H. Bates.

As the name indicates, this volume is concerned with the chemistry of cement and concrete. It devotes no more space to actual plant procedure in making cement or job procedure in making concrete, than to insure that the reader will realize something of the nature of the manufacturing methods involved. This statement does not carry with it the thought that the chemistry is so advanced that only an advanced thorough chemist would understand the subject as it is developed. On the contrary the authors have taken pains to present clearly, tersely, and adequately the fundamentals of the chemistry involved so that most engineers or enlightened cement mill chemists can readily follow the presentation.

About two-thirds of the book is given over to cement and the remainder to concrete. The portion concerned with cement contains an historical presentation of the development of hydraulic cements generally and portland cement in particular, followed by chapters on cement components and their phase relations, the cementing qualities of the compounds in cements, and the constitution of cement. These chapters are of particular interest in that they give the results of the senior author's recent work in the system lime-silica-alumina-iron and the discussions which he has very recently given on glass in portland cement. Two following chapters discuss manufacturing procedure and the reactions therein; then a chapter on the phenomena of setting and hydrating in water and in the presence of solutions of aggressive salts.

Puzzolana cements, cements from blast furnace slag, alumina or fused cements, and a group of "Special" cements are covered in five chapters. These should be of extreme interest to readers in the United States in view of the continued wide interest in cement of other than the portland type.

The chapters devoted to concrete cover the nature of aggregates of the various types, the resistance of concrete to natural destructive agencies—percolating water, frost, fire, and sea and other sulphate waters—and to such other materials as mineral and fatty oils, organic acids, sugar, sewage, etc. A final chapter is devoted to "post mortems" of concrete failures.

This volume fills an outstanding void in our literature of cement and concrete. There are many publications covering the making and using of these commodities, but in none of them has there been brought together, and collectively and orderly discussed, the developments of the last two or three decades in the physical chemistry of these materials. Realizing the immense importance of the findings of these researches, the authors have made their presentation with the background of physics and chemistry. Further, the authors realized that a considerable portion of their expected readers would not be sufficiently familiar with certain of the fundamentals of these subjects to permit of their interpreting and evaluating many of the facts

which they would present. Hence, they have given very briefly, simply and adequately, enough of the basic principles to enable the "nonphysical-chemist" to work out from the diagrams presented, if he so cares, such apparently intricate questions as the course of crystallization of melts of certain components from any assumed temperature.

Without much question some readers not too familiar with the status of our knowledge of what hydraulic cements are and how and why they react with water will be disappointed by the lack of positiveness at times or even with apparent contradictions of the authors. But our knowledge is at times most uncertain and at others the results of researches have been contradictory. The authors can hardly appear otherwise than uncertain. However, they have presented the best information at hand and discussed and evaluated it adequately.

The work is remarkably free from errors or misprints. Further, as publishers now seem to follow the practice of presenting rather shoddy work, both as to print and paper, to reduce costs, it is most pleasing to note the excellent quality of paper used and the general make-up of the volume. The senior author, F. M. Lea, will be recalled by many in this country. He spent a year here (1928-29), mostly in research, at the National Bureau of Standards and in travel through the country visiting cement plants and places where concrete work of interest was under way. The American Concrete Institute had the pleasure of his attendance at its annual meeting in Detroit in 1929.

Both authors and publishers are to be heartily commended for this volume, which is unquestionably the best of its kind on the subject.

In response to a spontaneous demand from Institute members for extra copies of the "Current Reviews" pages of each JOURNAL—to permit clipping up, pasting on cards and filing, arrangements have been made to supply to Institute members, (on request) in addition to their JOURNALS, two extra form proofs of "Current Reviews" pages of each issue at $1.50 per volume year. If the demand increases sufficiently it may be possible to reduce the charge for this extra service.

SECRETARY A. C. I.

Uniformity of Concrete on the Average Job—A Study of 13,000 Field Tests*

BY HUGH C. ROSS†
MEMBER AMERICAN CONCRETE INSTITUTE

Although durability and watertightness are important and sometimes deciding factors in the design of concrete mixtures, strength is usually a major requirement and strength tests, whether in compression or bending, are almost universally used as a criterion of concrete quality.

Generally, where a strength is specified, the concrete is designed from data established by preliminary tests of the materials available. The average job strength of the concrete will be in close agreement with that of the test data from which it was proportioned, but it is reasonable to expect that approximately half of the individual field tests making up this average will fall short of the strength requirements unless the mixture is designed for an average strength somewhat in excess of that actually specified.

The margin of strength provided above that actually specified might be termed the factor of safety or overdesign for assurance of quality, and the amount of overdesign necessary to meet given strength requirements depends upon the variability to be expected on the job. This has largely been determined by experience or judgment, but with the development of the ready-mix industry and the tendency towards purchase of concrete on a strength basis, there has been an urgent need for data that will serve as a guide in estimating the overdesign necessary to provide a satisfactory guarantee of quality.

Several investigators‡ have made valuable contributions to our knowledge of the strength variations that occur in the field, but in most cases the data have covered comparatively few tests from selected jobs. To obtain data more representative of average job conditions, an analysis has been made of the 28-day test results from 60 different job operations. Included in this paper, therefore, are data from the developments of the Hydro-Electric Power Commission

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‡See references appended to paper.

of Ontario, several ready-mix plants in the United States and Canada as well as some published test results. In all, the results of about 13,000 test specimens are represented.

DEVELOPMENT OF DATA

Variations in strength tests may be due to quality-changes in the concrete resulting from differences in materials, uncorrected variations in the moisture content of aggregates, errors in measurement and from methods of manipulation during manufacture. They may also result from conditions reflecting the amount of care exercised in sampling, molding, curing and breaking the test specimens. Some factors tend to produce strength variations proportional to the strength of the mix but others do not appear to have this proportional effect. These factors may occur in a great many combinations, and it is difficult to estimate the extent to which any one may be represented in a test or group of tests. The purpose of this paper is not to deal with these factors individually, but rather to investigate their combined effect upon the strength results of the average job.

It is customary to represent variations as percentages of the average, the percentage of tests falling within given limits of the average strength being used extensively as an index of job uniformity. In recent years, however, developments have led to the use of higher strength concretes which would make this a rather misleading method of rating job uniformity unless the magnitude and frequency of the variations are proportional throughout the range of strengths now being used. To have a uniform basis of comparison and to determine to what extent job uniformity is affected by the strength of concrete, the data used in this analysis were grouped under three general classifications for jobs having average strengths falling within the limits 1500-2500, 2500-3500, and 3500-4500 p.s.i. respectively.

In developing the basic data, field tests for each job were plotted as a progressive strength chart, a form used by many investigators in studying variations. Fig. 1 is a typical chart where the results of all specimens for a given job are arranged in order of magnitude with the abscissas showing percentages of the total number of tests falling below the strength indicated on the vertical axis. This arrangement gives a characteristic curve approximately straight in the middle and curved at both ends. The curved portions of the chart represent tests beyond the range of uniformity while those tests at the curve extremities represent abnormal or freak results. In general, the shape of the curve is fairly symmetrical about the center with approximately half of the specimens falling above and below the average job strength.

A chart of this type was drawn for each job and from it were listed the percentage of tests falling below the average strength and below various strengths higher and lower than the average. Seventeen such values were taken from each chart covering 100-lb. increments in strengths from 800 lb. below to 800 lb. above the average. These values were then weighted according to the number of tests represented by each job, and the weighted average values were obtained for the three general strength classifications under consideration.

Table 1 shows the weighted values listed in column 2 under each strength classification. Column 3 gives the average amounts that the individual jobs departed from these weighted values. For example, in the 3000-lb. classification the weighted results of 20 jobs show that 13 per cent of the tests deviated more than 500 lb. below the average strength or below 2500 lbs., while the mean variation of the 20 jobs from this 13 per cent figure was 6 per cent.

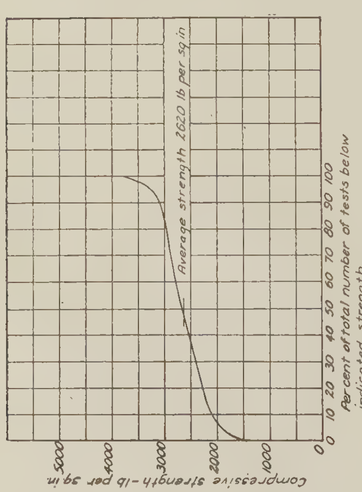
DISCUSSION OF DATA

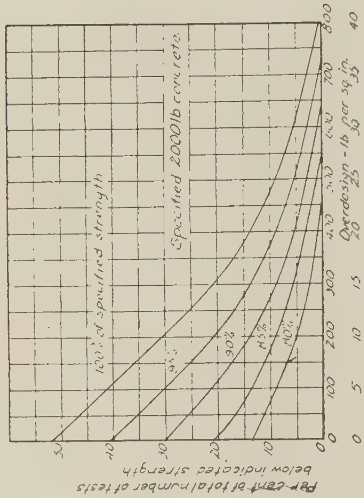
In Table 2, the data have been tabulated to show the number of tests falling within various percentage limits of the average strength. Included in this table is a 5000-lb. classification covering results of 1450 tests from 12 jobs reported by the University of Texas.* In this table it will be noticed that more tests fall within a given percentage limit of the average for high strength concretes than for low strength concretes. It would therefore appear that the magnitude of the variations are not proportional throughout the range of strengths now being used and that higher percentage variations may be expected in the low strength mixtures. This has also been observed in the laboratory where higher percentage variations are usually experienced in 7-day tests than in 28-day and 3-month tests molded from the same concrete.

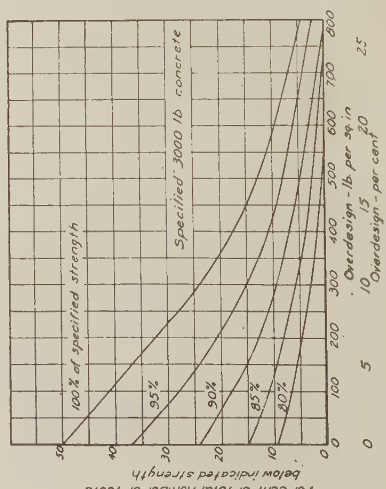
Overdesign of Mixtures—The data in Table 1 show that if no margin of strength were provided in designing a specified 3000-lb. concrete, about half of the test specimens would fall short of the strength requirements; whereas, if the mixture were designed for an average strength of 3200 lb., the number of tests below 3000 lb. would be reduced to 32.5 per cent. By increasing the amount of overdesign, the number of substandard tests would be reduced according to the average deviations shown in Table 1.

In Fig. 2, 3, and 4, the data have been more conveniently arranged to show how overdesigning will reduce the number of substandard

*John A. Focht—"A study of tests of Cylinders taken from Concrete Roads in Texas during 1928," University of Texas Bulletin No. 2922.







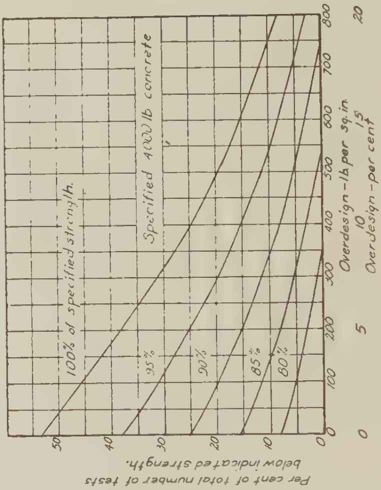


FIG. 1 (UPPER LEFT)

FIG. 2 (UPPER RIGHT)

FIG. 3 (LOWER LEFT)

FIG. 4 (LOWER RIGHT)

TABLE 1—DEVIATIONS FROM AVERAGE STRENGTH FOR THREE CONCRETE CLASSIFICATIONS

Average 2000-lb. Concrete 31 Jobs 2654 Tests			Average 3000-lb. Concrete 20 Jobs 2812 Tests			Average 4000-lb. Concrete 9 Jobs 6003 Tests		
(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
Compressive strength lb. per sq. in.	Average per cent of tests falling below strength indicated in Col. No. 1	Mean deviation of jobs from average value of Col. No. 2	Compressive strength lb. per sq. in.	Average per cent of tests falling below strength indicated in Col. No. 1	Mean deviation of jobs from average value of Col. No. 2	Compressive strength lb. per sq. in.	Average per cent of tests falling below strength indicated in Col. No. 1	Mean deviation of jobs from average value of Col. No. 2
1200	1.0	1.0	2200	4.5	3.5	3200	8.0	4.5
1300	2.5	2.5	2300	7.0	5.0	3300	11.5	5.5
1400	6.0	4.5	2400	9.5	5.5	3400	15.5	6.0
1500	9.0	5.5	2500	13.0	6.0	3500	19.5	7.0
1600	13.5	6.5	2600	18.0	7.5	3600	25.0	8.5
1700	21.0	6.5	2700	24.0	8.5	3700	31.0	9.0
1800	30.0	7.0	2800	32.5	7.0	3800	38.0	9.0
1900	41.0	7.5	2900	41.5	7.0	3900	46.0	8.0
2000	52.0	6.5	3000	50.0	7.0	4000	53.5	5.5
2100	65.5	6.0	3100	61.0	6.5	4100	62.0	4.0
2200	75.5	5.5	3200	70.5	6.0	4200	70.0	2.5
2300	82.5	5.5	3300	79.5	5.0	4300	77.0	1.5
2400	88.0	5.0	3400	86.0	4.5	4400	82.5	3.5
2500	92.5	4.5	3500	90.5	4.5	4500	87.0	3.5
2600	95.0	3.5	3600	93.5	4.5	4600	90.0	3.5
2700	96.5	3.0	3700	95.5	4.0	4700	92.5	3.0
2800	98.0	2.5	3800	97.0	3.0	4800	95.0	2.5

TABLE 2—PERCENTAGE OF TESTS FALLING WITHIN VARIOUS PERCENTAGE LIMITS OF AVERAGE STRENGTH

Limits of Average Strength, Per Cent	Per Cent of Tests Within Given Limits of Average Strength			
	Average 2000-lb. Concrete	Average 3000-lb. Concrete	Average 4000-lb. Concrete	Average 5000-lb. Concrete
± 5%	24.5	28.5	32.0	——
± 10%	45.5	55.5	57.5	59.5
± 15%	61.5	72.5	74.5	76.0
± 20%	74.5	84.0	87.0	87.5
± 25%	83.5	90.5	95.0	——

tests and what margin of strength should be provided to meet given quality requirements. These charts are not intended as specific data directly applicable to all job operations, but rather as average values derived from past job performances. It should be remembered that in some instances the individual jobs varied widely from the average values, and that although in some operations less overdesign might safely be used, conditions on other jobs might warrant greater over-designs than the charts indicate.

A typical example of the use of the charts might be taken for the case of a 2000-lb. concrete where strength tolerances stipulate that

not more than 20 per cent of the tests may fall below the specified strength and not more than 10 per cent of the tests may fall below 90 per cent of the specified strength. Referring to Fig 2, it is found that a minimum overdesign of 15.1 per cent would be necessary to satisfy both of these conditions.

A tolerance restricting the number of sub-standard tests has been included in the "Standard Specifications for Ready-Mixed Concrete"* of the American Society for Testing Materials. This specification requires that the average strength be equal to or above the specified strength, and that at least 90 per cent of all tests shall be equal to or greater than 90 per cent of the specified strength. To meet this requirement, the charts show that on the average job it would be necessary to overdesign 13.5, 9.3 and 8.5 per cent respectively for the three classifications being discussed in this paper.

Generally speaking, it has been considered good practice to overdesign mixtures by 15 per cent, a clause to this effect having been included in the 1928 Reinforced Concrete Building Regulations and Specifications† and in many city building by-laws. On this basis of design, data from jobs investigated in this paper show that the number of sub-standard tests for 2000, 3000 and 4000 lb. concrete would be 21, 15, and 15 per cent respectively, while the number of tests falling below a 90 per cent tolerance would be 9, 6, and 3 per cent respectively. A uniform percentage overdesign would result in more sub-standard tests in the low strength mixture, and therefore if the same tolerance were stipulated in each case, a graded percentage overdesign would be necessary to satisfy the requirements.

The 15 per cent overdesign recommended by the Concrete Building Code appears to be adequate for the average job where 3000 and 4000 lb. concrete are specified, but the data indicates that a somewhat higher overdesign is necessary for 2000-lb. concrete to give equal assurance of quality based on a percentage tolerance. A 20 per cent overdesign for 2000-lb. concrete, and a 15 per cent overdesign for the higher strength mixtures would substantially reduce the number of tests ordinarily falling below the specified strength if no overdesign were used, and would provide a safe margin of security for the guarantee of quality stipulated in the Standard Specifications for Ready-Mixed Concrete.

In considering strength requirements, cognizance should be taken of the fact that some substandard tests do not represent quality changes in the concrete. Concrete compression specimens are very

*A. S. T. M. Serial Designation C 94-35.
†*Proceedings*, American Concrete Institute, Vol. 24, 1928, p. 795.

sensitive to test conditions and due allowance should be made for this before passing final judgment on the quality of the mix.

Testing variations may be segregated into two general classifications, normal variations due to testing technique, and abnormal or "freak" variations due to faulty specimens. Normal testing variations are usually of comparatively small magnitude. This is indicated by the fact that on most well conducted jobs the average deviation between companion specimens molded from the same concrete is about five per cent or less. Abnormal or "freak" testing variations are rather infrequent in their occurrence, but they may result in deviations of considerable magnitude. This point should be recognized when considering quality requirements based on strength tests.

Examining Figs. 2, 3, and 4, it will be noticed that the curves fall very abruptly at first and then gradually flatten out showing that an overdesign beyond economic practicability is needed to ensure that all tests be equal to or greater than the specified strength. The engineer or architect should, therefore, be prepared to accept a certain percentage of substandard tests and should make provision for a reasonable number of low results due to faulty specimens as they are almost inevitable even on the most carefully controlled operations.

CONCLUSIONS

The average results of the 60 jobs analyzed show that low strength concretes are subject to greater percentage variations than high strength concretes and that a greater margin of strength should be provided in their design to ensure an equal quarantee of quality based on a percentage tolerance.

Attention is also directed to the fact that it is unreasonable to expect, as some engineers do, that all tests for a job be equal to or greater than the specified strength. The engineer should be prepared to accept a reasonable number of sub-standard tests and should make allowance for low values directly traceable to faulty test specimens.

Although the results of nearly 13,000 tests are represented in this analysis, it is not presented as being final or complete. It is hoped that similar data from a larger selection of jobs will be added from other sources. It is believed, however, that the data will be useful as a guide to those confronted with the problem of designing concrete to a standard of quality based on strength tests, and that it may serve as a reference for comparing the uniformity of job operations.

This study was made as a part of an investigation conducted by the Hydro-Electric Power Commission of Ontario to determine the quality of field concrete. Acknowledgment is made of assistance rendered by

the National Ready-Mixed Concrete Association in canvassing its members for data included in the paper.

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For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Concrete at Norris Dam*

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Norris Dam is being built by the Tennessee Valley Authority as part of the flood control, navigation and power project now in progress in the Tennessee River basin. The dam is on the Clinch River approximately 25 miles northwest of Knoxville, Tenn., immediately below the mouth of Cove Creek. The reservoir above the dam will impound 3,600,000 acre feet of water in a lake having two forks each 60 miles long. The dam is a concrete gravity structure 1600 ft. long and 254 ft. in maximum height, with a concrete corewall extending into the east abutment. A power house, containing two 66,000 horsepower units, is being built at the toe of the dam on the east bank of the river. The total amount of concrete in the dam, corewall, power house and appurtenant structures will be slightly more than 1,000,000 cu. yds.

AGGREGATES FOR CONCRETE

The problem of producing 1,000,000 cu. yds. of concrete, with which this paper deals, was complicated by the absence of suitable natural aggregate in sufficient quantities within reasonable distance of the damsite. However, the entire region consists of a generally sound dolomite rock covered, in most places, by a comparatively shallow layer of clay soil, and when investigation failed to show the presence of natural aggregates consideration was given to the possible use of aggregate, including sand, which could be made from this abundant material. After careful consideration of the properties of the dolomite by engineers of the Authority and extensive tests at the Denver laboratories of the U. S. Bureau of Reclamation, a decision was reached, which called for quarrying and crushing 2,000,000 tons of dolomite rock to furnish aggregate for the estimated 1,000,000 cu. yds. of concrete.

Manufactured aggregate offered an advantage in uniformity over natural aggregate, but because of particle shape and poor grading in

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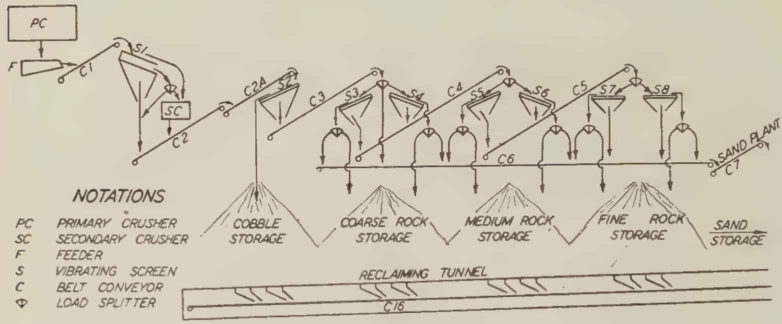

FIG. 1—FLOW SHEET—LARGE AGGREGATE

the smaller sand sizes a high cement requirement was expected for its use in concrete. The aggregate plant for Norris Dam was designed to take advantage of the uniformity and to minimize the effect of poor particle shape and poor grading in the fine sizes. Provision was made for producing six sizes of aggregate. A brief description of the quarry and of the coarse aggregate crushing and screening plant where four sizes of material are produced will be given. A more detailed description of the sand plant, producing two sizes of aggregate, where the major problems were found, will follow.

The quarry is 2000 ft. upstream from the west abutment of the dam on a side hill approximately parallel to and facing the dam. The dolomite lies in nearly horizontal layers 2 to 15 ft. thick, separated in most cases by fairly tight seams carrying negligible amounts of clay. Except for small amounts of weathered material, the rock is very uniform in its physical and chemical properties. Quarry overburden, a very tough red clay, was stripped hydraulically as completely as possible before the actual quarrying began, but due to a very irregular rock surface some overburden was left and was wasted as quarry operations progressed. An approximately straight vertical face 1200 ft. long was developed and extended 250 ft. into the hillside, furnishing more than half of the required aggregate. The remainder is being produced by benching below the original quarry floor in two 28 ft. lifts. The dolomite is removed from the quarry by light blasting in wagon drill holes, loaded into trucks by electric shovels and hauled to the primary crusher.

The flow sheet (Fig. 1) shows sequence of operations in the large aggregate crushing and screening plant, and requires little comment. The four sizes of aggregate produced are deposited in separate stock piles over a concrete reclaiming tunnel. Nominal sizes are cobbles,

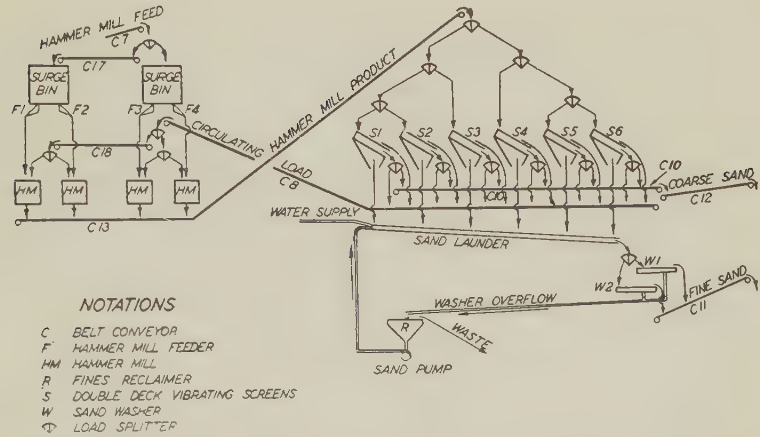

FIG. 2—FLOW SHEET—SAND PLANT OPERATION

3 to 6 in.; coarse rock, 1½ to 3 in.; medium rock, ¾ to 1½ in., and fine rock ⅜ to ¾ in. No serious problems were found in the aggregate plant, except as might be expected in the separation of the fine rock from quarry fines. Screen cloth of not less than ⅜ in. square opening was found necessary for obtaining clean screening at this point in wet weather.

The sand plant consists essentially of four hammermills, six vibrating screens, two sand washers, one classifier for very fine material and the necessary equipment for storing and transporting the various materials. It produces two sizes of sand, one finer than 8 mesh and the other between 8 mesh and ⅜ in., and delivers them to separate storage piles over the reclaiming tunnel previously mentioned. Hammermills and screens are operated dry, as it was found that wet operation greatly increased hammer wear and added moisture control difficulties at the mixing plant. The flow sheet (Fig. 2) indicates the general scheme of sand plant operation.

Hammermills were selected for sand production after studying workability of sand-cement mortars containing dolomite sand produced by several types of equipment. Low cement requirement for a given strength, apparently due to particle shape of the hammermill product, was a deciding factor. Dolomite breakage curves obtained from various types of sand mills showed approximately equal amounts of waste in producing sands of similar grading. Hardness of the dolomite is such that it may be reduced in hammermills without excessive metal wear, although the silica content ranged from 4 to 7 per cent.

As shown on Fig. 1, feed for the sand plant may be obtained from any size aggregate except cobbles, providing without waste material for sand production under all conditions of coarse aggregate plant operation. Surge bins of sufficient capacity for one hour of sand production are provided. Hammermill product is screened by six double decked screens having 8 mesh cloth on the lower decks and 7/16 in. cloth on the upper decks. Material through the lower decks is washer feed. Reject from the lower deck is coarse sand, part of which is stockpiled and the remainder returned to the mills as circulating load. All reject from the top deck screens is circulating load.

Importance of controlling the finer sand sizes required the addition to the plant as originally designed of a classifier (noted on the flow sheet as "fines reclaimer"). Into this apparatus the washer overflows, containing large amounts of fines (—100 mesh), are discharged. The coarsest fraction of these fines is settled out and returned to the sand launder as indicated. Very fine material is overflowed and wasted. By varying the amount of circulating load carried through the reclaimer and the amount of fresh water added to the system practically any desired amount of —100 mesh sand may be retained in the sand produced by the washers. Waste is largely undesirable "slimes" finer than 325 mesh. The portion retained is fine sand between 325 and 100 mesh.

Aggregates are drawn from the stock piles through gates in the roof of the reclaiming tunnel. Fine sand, stock piled wet in a long pile, is admitted to the tunnel by any one of eight gates equally spaced under the sand pile, permitting selection from well drained portions of the pile. Two gates are provided for each of the other five sizes. A belt conveyor, operating through the length of the reclaiming tunnel, removes aggregates one size at a time to a second belt conveyor leading to storage bins at the top of the mixing plant structure. A manually operated turnhead deflects the conveyor discharge into the proper bin.

CONCRETE PRODUCTION AND PLACING

Cement is shipped in rebuilt box cars from one of three cement mills to an unloading station 4½ miles from the dam. It is unloaded by portable cement pumps discharging into a 6000-bbl. steel storage silo. Transportation from the silo to the dam is by tank trucks of 65-bbl. capacity each. Trucks are unloaded by gravity into hoppers near the mixing plant, from which the cement is pumped to either of two 500-bbl. mixing plant bins or into storage in a second 6000-bbl. silo nearby. Cement may be pumped from this storage silo to the

mixing plant bins by the pumps which are used to fill the silo. The total 13,000 bbl. cement storage is sufficient for three days of maximum concrete production.

The mixing plant contains three 3-cu. yd. double cone tilting mixers, arranged symmetrically around a conical open bottom discharge hopper. The mixers are supported by a reinforced concrete structure. The remainder of the plant is carried independently on a structural steel framework separated from the mixer support to decrease undesirable vibration. One set of batchers and one hopper is made to serve all of the three mixers by means of a three-position motor driven turnhead, which deflects the batched materials through downspouts to the charging end of the desired mixer. Water is carried in one spout and all other material in a second larger spout. Eight bins, one for each size of aggregate and two for cement, are built into the top of the structure, providing storage sufficient for 600 cu. yds. of mass concrete. Aggregates are batched by individual manually operated weigh batchers, provided with dial scales. Duplicate cement batchers, with both dial and beam scales for either manual or automatic operation, are provided. Water is batched automatically by a displacement batcher. Mixer timers with interlocking devices prevent dumping of mixers in less than the required 2½ minute mixing time.

Equipment for control and dispatching of concrete is located in a small room on the batching floor of the mixing plant overlooking the dam. A switchboard permits the dispatcher to light signal lamps, attached to movable pointers on the dial faces of the batcher scales. The pointers indicate desired weights. Five sets of lights with pointers are provided to give five different concrete mixes as desired, without resetting of scales pointers. The water batcher is set by the dispatcher by means of push buttons. Recording watt meters for each mixer, batch counters for each of the five mixes, signal lights showing turnhead position and a volt meter indicating moisture content of fine sand are included on the switchboard. An electric oven, balances, etc., are provided for moisture determinations on aggregates.

A dispatcher and a plant inspector and the necessary men to do the weighing and to operate the mixers, are responsible for control and proper disposition of all concrete produced by the plant. Telephoned orders for concrete for perhaps an hour in advance, requiring two or more mixes alternating for different locations in the dam, are received and executed without confusion.

Uniformity of concrete has been appreciably improved by use of the electrical apparatus for measurement of moisture in fine sand. This apparatus, checked frequently by oven dried sand samples, provides

means for correcting mixing water for sudden moisture changes in the fine sand. The necessary re-setting to the water batcher can usually be made before the batch showing change is in the mixer. The electrical apparatus is used only for moisture indications between determinations made by oven drying.

Transfer trains, operating on standard gage track, transport concrete from the mixing plant to 6 cu. yd. cableway buckets spotted on a landing platform below the level of the transfer track. Length of haul varies between 300 and 750 ft. Each train consists of a gasoline-electric locomotive and a car with two 3-cu. yd. tilting hoppers, a central side discharge hopper and a short section of retractable chute for directing concrete into the cableway bucket. Remixing during discharging through the center hopper appreciably reduces segregation of cobbles for normal mixes, and filling of buckets has little effect on segregation except for extremely dry concrete.

Two 18-ton cableways of 1900-ft. span suspended from movable steel towers serve all parts of the dam, spillway apron and power house. Each cableway handles one 6-cu. yd. concrete bucket between the landing platform and the forms in cycles of 2½ to 5 minutes, depending on location of the point of deposit.

Mass concrete is placed in 5-ft. lifts in construction blocks of 56-ft. width and of length equal to dam thickness. Lifts are sloped 5 per cent downward toward the upstream face of the dam. Placing of concrete is preceded by application of cement-sand mortar ½ to 1 in. thick thoroughly brushed into the old surface with wire brooms. The old concrete surface is dampened before receiving the mortar and the mortar is covered by concrete as soon as possible to prevent its drying out. In general, concrete placing begins at the upstream face and progresses downstream, against the 5 per cent slope. Completed portions of a lift placed in this manner may be cleaned without disturbing concreting operations which may still be in progress farther downstream in the same block.

After a cableway bucket has been lowered to its desired position, approximately 2 ft. above the concrete already in place, the gate in the bottom is opened by means of compressed air applied through a small hose with valve and suitable end fitting to a receptacle on the bucket. Opening the valve forces compressed air to cylinders built into the bucket which release safety latches and operate the heavy dumping mechanism. The bucket is self-closing and self-latching and requires no hand manipulation except occasional cleaning of accumulated concrete which may prevent automatic closing of the bucket.

Development of the compressed air dumping apparatus, which was accomplished on the job, has greatly increased the speed and safety of cableway concrete placing.

Each 6-cu. yd. pile of concrete is vibrated into place by means of internal vibrators driven by three phase electrical power. Surface vibrators compact the top surface of each lift. Frequency changers with V-belt drives and interchangeable pulleys for obtaining various frequencies above the 60 cycles available were installed as part of the vibrating equipment with the idea that the highest feasible frequency would be used. Beginning with 68 cycles, frequencies have been increased by steps to 80 cycles without serious increase in maintenance of vibrating equipment. The 80-cycle frequency was adopted and has been maintained throughout the job. It is believed that the increase in vibrating efficiency due to the change from 60 to 80 cycle power (3600 to 4800 vibrations per minute) makes possible the use of 0.05 bbl. less cement per cubic yard in the mass concrete.

Particular attention is given to preparation of horizontal construction joint surfaces to which following concrete lifts must be bonded. The top film (very thin for dry concrete) is removed by a jet of air and water forced together through a nozzle and directed against the surface. By proper application of the jet at the proper time a clean sound surface is obtained without disturbing the partially set mortar or loosening the larger rock particles. Material removed by this process is washed down the 5 per cent slope and wasted over the upstream face of the dam. At least one additional washing is given each surface after form and other work for the following lift has been completed. No keyways or other form of roughening is provided in the horizontal joints.

Maximum production of the mixing plant under the 2½-minute mixing time requirement is 180 cu. yds. per hour, and after development of operating technique it has been possible to produce concrete at that rate for continued periods. Damp aggregates appreciably increase the time required for charging of mixers and reduce the plant output. Since development of the compressed air dumping apparatus for cableway buckets during the early part of the job, the cableways have been able to handle concrete faster than the mixing plant can produce it. Under normal full capacity operation 3600 cu. yds. of mass concrete were placed during each 23-hour working day and a maximum of 4100 cu. yds. were placed in one 24-hour period.

TECHNICAL DATA

A field laboratory equipped for routine testing and experimental work is located near the mixing plant. Testing equipment and a small

office are housed in a one-story temporary building of frame construction 24 x 50 ft. Aggregate bins for materials passing each standard screen from 200 mesh to 6 in. are provided. Equipment includes a small tilting concrete mixer, aggregate screening machine, 300,000 lb. testing machine and a 70° F. constant temperature fog foom having a storage capacity of 1500 6 x 12 in. test cylinders. Special equipment for temperature and strain measurements on laboratory test specimens and for use in the dam is provided.

Standard practice is observed for routine testing, except that concrete is vibrated into cylinder molds by a small internal vibrator. Practically no difference in strength between this and the standard rodding method has been observed for workable concrete mixes. Concrete containing large aggregate is wet screened to 1½-in. maximum size and tested in 6 x 12-in. cylinders. In compression tests a constant rate of loading of 17 p.s.i. per second is used.

Cement is a modified portland, designated as type B, with 8 per cent maximum C_3A, C_3S between 35 per cent and 55 per cent and specific surface between 1600 and 2200 square centimeters per gram (by Wagner turbidimeter). Three producers manufacturing cement under specifications, including the items mentioned, furnish products of uniformly high quality. In the cement received C_3A averages 6 per cent, C_3S varies between the limits of 35 and 55 per cent, and fineness is uniform with a specific surface ranging between 1750 and 1950 sq. cm. per gram. Early strengths, particularly for cement near the low C_3S limit, are somewhat lower than for normal portlands, but in all cases have been entirely adequate for construction requirements. No requirement for heat of hydration was included in the cement specifications, but temperature rise in the dam and tests by the Bureau of Reclamation Laboratories indicate reasonable uniformity, with values ranging between 75 and 85 calories per gram at 28 days.

Three general types of concrete are used in the dam. The interior portion of the dam contains mass concrete having 0.9 bbl. cement per cu. yd., a water cement ratio (w/c) of 0.67 (by weight) and 6 in. maximum size aggregate. Mass concrete comprises the bulk of all concrete produced. Exposed surfaces of the dam contain "face" concrete, with 1.20 bbl. cement per cu. yd. and w/c of 0.56 for spillway surfaces and 1.10 bbl. cement per cu. yd. and w/c of 0.58 for other faces, both using 6 in. maximum size aggregate. Concrete for reinforced sections with maximum aggregate sizes of 3 in. or 1½ in. and 1.33 bbl. or 1.50 bbl. cement per cu. yd. uses w/c of 0.55. Mortar used to cover construction joint surfaces contains both sizes of sand,

2.8 bbl. of cement per cu. yd. and has a water ratio of 0.53. Special mixes are used when necessary, but those noted above fit most conditions encountered. Consistency of the concrete mixes noted varies between 3 in. slump for the reinforced concrete and 1 in. slump for the mass concrete. Water ratios remain constant, cement and water together being varied when necessary to produce required workability.

As soon as concrete placing in the dam was started it became evident that control of aggregate grading was to be of utmost importance from both quality and economy standpoints, with particle sizes smaller than the 8-mesh standard screen assuming the most importance. The breakage curve for dolomite fixed the available amounts of particle sizes between 100-mesh and 8-mesh within narrow limits, unless radical changes were made in the sand screening plant. Therefore attention was directed to the particles finer than 100 mesh. As a result the "reclaimer" previously described was installed to recover fines in the overflow from the sand washers. Uniformity of grading was greatly improved and the "slimes," largely —325 mesh, considered undesirable, could be wasted as before. The amounts of —100 mesh fines were raised from 11 to 17½ per cent of the fine sand (from 7 to 12 per cent of the —4 mesh sand) the proportion found to give best results in the concrete. The change produced a more workable concrete with greatly reduced water gain, and finally permitted a reduction of cement content in the mass concrete from 1.00 bbl. per cu. yd. to 0.90 bbl. per cu. yd. without change in water ratio. The most desirable grading of aggregate sizes larger than 8 mesh was determined by close observation in the field, the final result aggreeing reasonably well with recent grading theories with which it has been compared. Flexibility of the aggregate plant permits any reasonable combination of the six aggregate sizes, so, in general, the most desirable grading is also the most economical. Uniformity in fineness of cement eliminates variations in concrete workability chargeable to cement. Grading of the combined aggregate and mix data for mass concrete are shown on Fig. 3.

Compressive strengths (28 days and longer) of all mixes are appreciably higher than might be expected from the water-cement ratios employed. Laboratory tests on other aggregates indicate that the high strengths are due largely to the dolomite aggregate, though the type B cement has been shown to have higher than normal strength possibilities with all aggregates. Strengths of mass concrete and face concrete mixes, as shown by 6 x 12 in. test cylinders of concrete wet screened to 1½ in. maximum size, are shown on Fig. 4. Corresponding strengths of large cylinders containing the full mixes may be approximated by multiplying values shown by eight tenths.

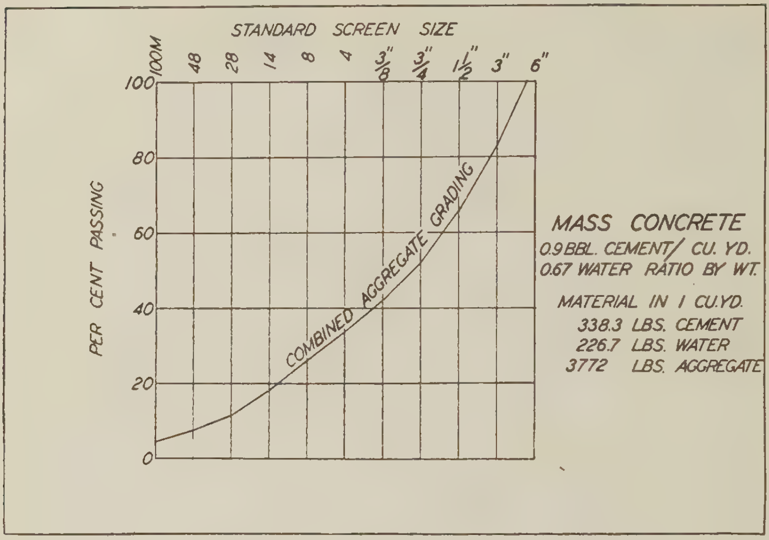
STANDARD SCREEN SIZE
100M
48
28
14
8
4
3/8"
3/4"
1 1/2"
3"
6"
PER CENT PASSING
100
80
60
40
20
0
COMBINED AGGREGATE GRADING
MASS CONCRETE
0.9 BBL. CEMENT/ CU. YD.
0.67 WATER RATIO BY WT.
MATERIAL IN 1 CU.YD.
338.3 LBS. CEMENT
226.7 LBS. WATER
3772 LBS. AGGREGATE

FIG. 3

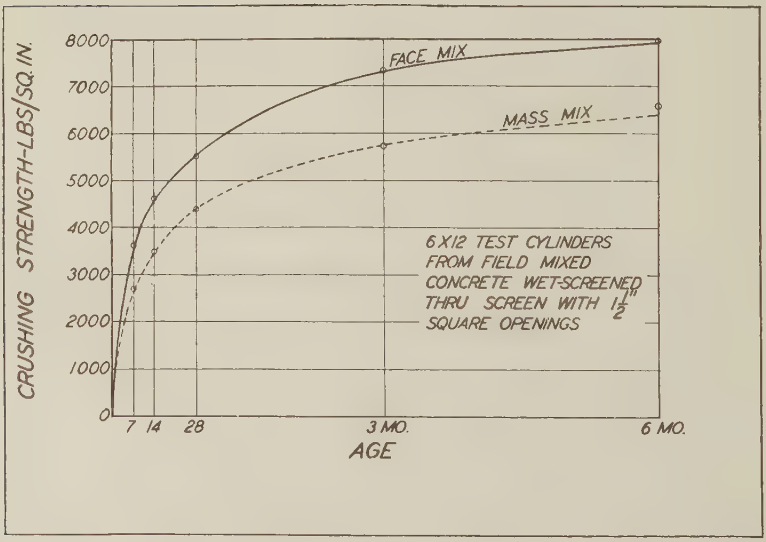
CRUSHING STRENGTH-LBS/SQ.IN.
8000
7000
6000
5000
4000
3000
2000
1000
0
FACE MIX
MASS MIX
6 X 12 TEST CYLINDERS
FROM FIELD MIXED
CONCRETE WET-SCREENED
THRU SCREEN WITH 1 1/2"
SQUARE OPENINGS
7
14
28
3 MO.
6 MO.
AGE

FIG. 4

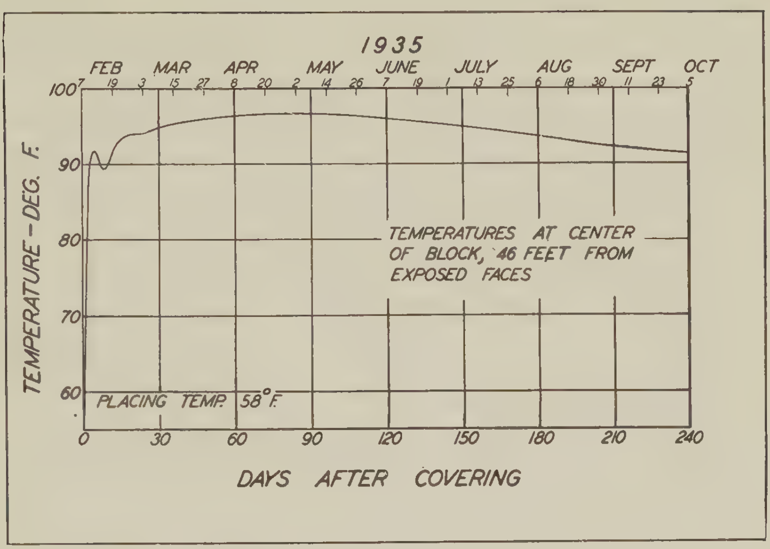

Fig. 5

One construction block of the dam is being placed using a mixture of type B cement and blast furnace slag cement in a proportion of 3 to 1 (volume measure), 25 per cent of the type B cement used in other parts of the dam being replaced by an equal volume of slag cement. The two cements are batched at the mixing plant by the separate weigh batchers and mixed during the 2½ minute period required for all concrete. Placing conditions in the block containing the mixture are identical with conditions found elsewhere in the dam, and its placing schedule is identical with that of one other block in the dam, so excellent opportunity for comparisons exist. Investigations of strength, temperature rise, and other characteristics of concrete in the special block are being made for comparison with that in other parts of the dam. Durability comparisons may be made in future years by observation of exposed surfaces subjected to identical weathering conditions, spillway overflow erosion, and any other conditions which may affect disintegration of concrete.

Special measurements in the dam are being made on temperature, strain, contraction joint openings and hydraulic uplift. Except for temperature measurements, most of such work is still in a preliminary stage. Approximately 150 resistance thermometers are located in various parts of the dam for determination of temperature rise, rate of cooling, average temperatures, etc. A typical time temperature

curve for mass concrete in thick sections of the dam is shown on Fig. 5. Carlson elastic wire strain meters, located near the foundations of two high blocks, were installed to show conditions of strain in this part of the structure, with the hope that the results might be of value in determining stress distribution at the foundations. Three stations in each block are arranged to measure plane strain at right angles to the axis of the dam and to give one strain measurement parallel to the axis. "No stress" meters in small recesses nearby are used for comparisons.

TABLE 1—AVERAGE STRENGTH OF FIELD MIXED CONCRETE

November 30, 1935

Mass Concrete

Note—All concrete having aggregate larger than 1½ in. was wet screened through 1½-in. square opening wire screen. All cylinders are 6 by 12 in.

Rate of loading—17 lb. per sq. in. per second.

Bbl. per cu. yd.	1.00		0.95		0.90	
W/C	0.67		0.67		0.67	
Age	No. Cylinder	Average Strength	No. Cylinder	Average Strength	No. Cylinder	Average Strength
7 days	74	2529	354	2689	228	2695
14 days	23	3207	185	3599	178	3485
28 days	68	4047	306	4483	224	4395
90 days	60	5488	349	5816	238	5739
6 mo.	23	6073	169	6397	36	6624
1 yr.	27	6513	56	6790		

Face Concrete

Bbl. per cu. yd.	1.20		1.20		1.20		1.10	
W/C	0.60		0.57		0.56		0.58	
Age	No. Cylinder	Average Strength	No. Cylinder	Average Strength	No. Cylinder	Average Strength	No. Cylinder	Average Strength
7 days	46	2833	98	3617	33	3432	60	3522
14 days	21	3810	51	4646	33	4485	57	4426
28 days	48	4730	82	5500	45	5453	99	5360
90 days	66	6328	104	7332	43	6801	86	6867
6 mo.	21	7057	40	7962				
1 yr.	38	7697						

Reinforced Concrete

Bbl. per cu. yd.	1.50		1.33	
W/C	0.55		0.55	
Max. Agg.	1½ in.		3 in.	
Age	No. Cylinders	Average Strength	No. Cylinders	Average Strength
7 days	25	3329	19	3262
14 days	21	4386	13	4425
28 days	30	5133	22	5493
90 days	32	6490	18	7053
6 mo.				
1 yr.				

Elastic wire joint meters, installed across contraction joints in the west portion of the dam, are expected to furnish a history of joint openings, which will be of particular value in connection with grouting of the joints. Thermometer, strain meter, and joint meter leads are brought to permanent terminal boards in inspection galleries for convenience in reading. Uplift cells for measuring hydrostatic head at various locations on foundations and throughout the dam are being installed, with pipe connections leading to accessible locations in the dam.

Conclusions drawn from construction work and from laboratory testing at Norris Dam, which appear to be of particular significance, include the following items:

It has been shown that concrete of exceptional quality may be made with crushed dolomite aggregate, including the sand, of proper gradation.

The importance of particle size gradation and particle shape in the finer sizes of aggregate has been made particularly apparent.

Uniformly high fineness of cement has proved to be of more than usual importance.

The value of vibration in placing of concrete containing manufactured aggregates is even more evident than when natural aggregates are used. High vibrating speeds are very desirable.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

A Method for Determining the Air Content of Freshly Mixed Mortars and Concretes*

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Designers of concrete mixtures have not generally regarded air as one of the regular ingredients of concrete, partly for the reason that its determination is more or less uncertain, but mainly, no doubt, because the amount of air in plastic mixtures is assumed to be negligible. However, it is a fair question to ask: How much entrained air should be considered negligible? Probably most engineers will answer: Not over 1 per cent. But 1 per cent of entrained air in a cubic yard is .27 cu. ft., and according to the void-cement ratio theory, this has the same effect on strength as 2 additional gallons of water. In concretes containing from 5 to 7 sacks of cement per cubic yard, 1 per cent of air therefore has the effect of increasing the water content by .3 to .4 gallons per sack. For close design and control this is too large an error to be neglected, and it will be seen from some of the tests reported herein, that this amount of air, and more, is very likely to be entrained in this type of concrete.

But the question assumes even more importance in the case of mortars of the variety commonly used in testing, where upwards of 3 per cent of air is the rule rather than the exception. In the type of mortar now tentatively specified by the A. S. T. M. for compressive strength (C109-34T), the void-cement ratios for 7 standard portland cements, all gaged to a w/c of 0.8, were found to vary from 0.96 to 1.02, indicating air contents from about 5.5 to 7.5 per cent.[1] In mortars of this sort the air content is so great that one would expect it to vary with type and fineness of cement, and to some extent with manipulation, and one might wonder whether the properties of these mortars as indicative of cement quality necessarily reflect those of concrete of a corresponding w/c ratio with low air content. A few

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[1]Report of the Working Committee of Committee C-1 on Plastic Mortar Tests. Proc. A. S. T. M., Vol. 34, Part I, pp. 322-355.

data are included in this paper to show the change in air content of the standard plastic mortar with change in the type of cement.

Without over-emphasizing the possible effects of entrained air on other desirable properties of concrete, one should not forget that shrinkage, absorption, permeability and durability are not benefitted by air voids. With reference to shrinkage Carlson[2] says: "A further effect of the dilution of paste with aggregate will be the entraining of air and the production of voids not present in the neat paste. The voids not only permit the more ready escape of moisture from the mortar or concrete, but they may also permit the paste to shrink more freely." In studies of the permeability of concrete for Boulder Dam, Ruettgers, Vidal and Wing[3] discussed the effect of air voids, which in this type of concrete were stated to be 1.1 per cent of the absolute volume of the concrete, nearly 10 per cent of the water voids. Whether air voids contribute more or less than their proportionate share to higher permeability and lower durability may depend upon their size and distribution, but one gets the impression that in freezing and thawing tests of concrete, and particularly of mortars, some definite knowledge should be had of voids, or of void-cement ratios, as well as water-cement ratios.

METHOD AND APPARATUS

The contribution of the authors to the problem of air determination is a comparatively simple and direct method which largely avoids the chief sources of error in the prevailing method. In the latter, the specific gravities of the concrete ingredients are rarely known with sufficient accuracy to warrant the computation for air voids; there is also a shrinkage in the absolute volumes of cement and water brought about by their chemical reaction, which, even when the specific gravities are accurately determined, tends to give too low values for the computed air content. The proposed method is not inherently new—it is merely an adaptation of a pycnometer with certain unique features and a procedure which contributes to the accuracy and fool-proofness of the determinations. Since the problem consists fundamentally in determining the difference between the bulk and absolute volumes of a mixture, and since this difference is usually small compared to the volumes themselves, it involves the determination of relatively large weights with considerable accuracy. An essential piece of equipment is therefore a good balance, for which a Troemner No. 80 solution

[2]R. W. Carlson. The Chemistry and Physics of Concrete Shrinkage. Proc. A. S. T. M. Vol. 35, Part II, 1935, pp. 370-379.

[3]Arthur Ruettgers, E. N. Vidal and S. P. Wing. An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam. JOURNAL American Concrete Inst., Mar.-Apr. 1935, *Proceedings* Vol. 31, p. 382-416.

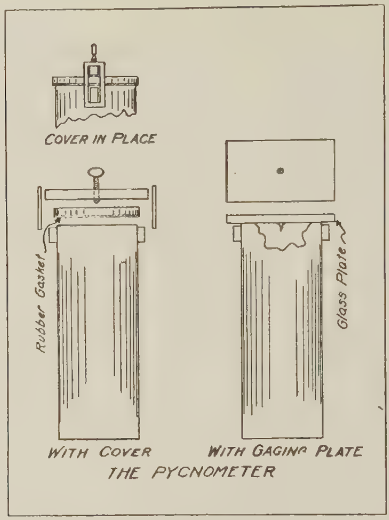

FIG. 1—PYCNOMETER, OR WEIGHING POT, WITH ACCESSORIES

scale, with a capacity of 20 Kg. and a sensitivity of about .5 g., has been found satisfactory. The pycnometer is a cylindrical brass pot made of a piece of drawn brass tubing 5 in. in diam. and 1 in. high as shown in Fig. 1. The auxiliary equipment consists of a gaging device to determine the position of the water level when the pot is filled, simply constructed of a piece of plate glass having a 1-in. brass screw filed to a sharp point and firmly affixed to the mid point of the plate with sealing wax. The pot also has a water-tight cover with a quick acting clamp arrangement, as indicated in the figure.

PROCEDURE

The method of operating involves making up a batch of mortar or concrete, preferably somewhat greater than 0.1 cu. ft. in volume, from which, after mixing, a 0.1, cu. ft. measure is filled, say, by the standard method of rodding 25 times in 3 layers. The weight of the unit bulk volume of the mixture is thus determined. The concrete is then returned to the mixing pan, remixed, and a portion of it is scooped into the pot, sufficient to fill the latter to a depth of 8 in. or 9 in. The weight of this concrete is then determined, and its bulk volume can later be computed by comparison with the weight of the concrete in the 0.1 c. f. measure. The pot is now filled with water approximately to the level of the gage point, the cover clamped on, and the concrete thoroughly mixed with the water by first turning the pot end over end, and then by rolling back and forth on the edge of a table. A few

trials will indicate how much of this turning and rolling is necessary to remove completely the air from the mixture, but there is no guesswork involved in the process when the final determination is made. When the mixture is judged to be entirely freed from air, the pot is slowly up-ended, the cover removed and adhering solids washed back into the pot by a dash of water from a beeker. Foam must be skimmed off the surface of the liquid, but care should be taken that no appreciable amount of fine cement or silt is removed with it. The pot is now placed on a firm and accurately level table, and the water brought to the level of the gage point. This is determined by the jump of the meniscus when the gaging device is lowered into place without jar. With a little care, the level of the liquid can be determined within a few thousandths of a millimeter, or in terms of volume, well within .5 cc. of water. When the gaging is completed, the pot is cleaned and dried on the exterior, and the weight of pot and contents determined.

To insure that the air is completely removed, the pot is again sealed with the cover, and the operation of rolling and redetermining the water level is again carried out. If the weight of the system has not then increased by more than 1 or 2 grams, the experiment is concluded by determining the temperature of the water to about 0.1° C. If the gain in weight is more than 2 or 3 grams, the rolling and regaging should be again repeated.

The weight of the water to fill the pot follows immediately from the total weight as finally determined. The weight of this water divided by its density (taken from the Smithsonian or similar tables, corrected if necessary by density determinations of the water used) gives its volume, and this volume subtracted from the predetermined volume of the pot (up to the gage point level) gives the absolute volume of the concrete. Finally, the computed bulk volume of the concrete in the pot, minus the absolute volume, gives the volume of entrained air, which may be expressed as a percengage of the bulk volume.

TEST DATA

Determinations of the Volume of Pot and Density of Tap Water. The order of accuracy to be expected in the gaging and weighing operations is indicated by the data in Table 1. The volume of the pot was determined by twenty weighings of distilled water, two on each of 10 different days. Corresponding to the observed temperatures, the densities were taken from the Smithsonian tables, and the volumes computed therewith. On the same days twenty weighings were also made of the pot full of tap water, and the temperatures noted. After

the series was finished and the volume of the pot had been fixed at 4104, the densities given in Table 1 were computed by dividing the weights by this volume. The average density and the average temperature were then derived, and the density of distilled water at this temperature was taken from the tables. In this manner the density of the tap water was found to be approximately .0004 greater than that of distilled water. In subsequent operations with the pot, in which tap water was regularly used, a correction of +.0004 was always applied to the values from the tables.

TABLE 1—MEASUREMENTS OF POT VOLUME AND DENSITY OF TAP WATER

	Volume of Pot, cc.		Density of Tap Water		Temp. of Tap Water, °C.
	1	2	1	2	
1	4104.2	4104.2	.99824	.99860	20.8
2	4103.8	4103.9	872	872	19.7
3	4104.8	4104.3	848	884	20.0
4	4104.7	4104.2	860	860	20.6
5	4103.1	4103.1	824	860	20.4
6	4103.1	4103.1	884	897	19.35
7	4103.3	4102.8	836	848	20.0
8	4102.4	4101.9	836	836	19.9
9	4104.3	4104.3	884	872	20.15
10	4104.1	4105.1	909	897	19.95
Means	4103.8	4103.7	.99858	.99868	20.085
Avg.	4104		.9986		20.1

Density of Distilled Water at 20.1° C., .9982.
Correction for Density of Tap Water +.0004.

From the above table it is seen that the individual determinations of the pot volume are in most cases within 1 cc. of the average volume. It is believed that the larger variations arise from the uncertainty in weighing rather than in gaging the liquid level.

Reproducibility of Air Determinations. The procedure in determining entrained air, described in preceding paragraphs, is illustrated in Table 2, which gives the full data of check tests on four similar batches of concrete. This is the type of concrete used regularly in the testing of cements in our laboratory, the characteristics of which are as follows: Max. size of aggregate ¾ in.; slump 5 in. to 7 in.; net water-cement ratio 0.8; cement factor about 6.2 sacks per yard, varying slightly with the water requirements of different cements. The aggregate grading is given in Fig. 2B, Curve VI. The sequence of items in the first column of Table 2 is a time-saving guide for securing both observed and computed data, the former being indicated by italics, the latter by regular type.

It will be seen from Table 2 that the repeat determinations of entrained air are in good agreement, and that the computed specific gravities are remarkably uniform. It may be stated that the concrete

TABLE 2—REPRODUCIBILITY OF AIR DETERMINATIONS IN CONCRETE

	(Weights in grams, Volumes in cc)			
1. *Weight of Cement*	1150	(Same for all batches)		
2. *Weight of Fine Aggregate*	2275			
3. *Weight of Coarse Aggregate*	4225			
4. *Weight of Water*	680			
5. Weight of Batch, (1) + (2) + (3) + (4)	8330			
6. *Weight of .1 c. f. Measure* (V =2830 cc.)	4232.5	4232.5	4232.5	4232.5
7. *Weight of .1 c. f. Measure + Concrete*	10908.5	10912.0	10907.5	10913.0
8. *Temp. of concrete, °C*	19.9	19.5	19.8	19.8
9. Weight of Concrete, (7) — (6)	6676	6679.5	6675.0	6680.5
10. *Weight of Pot (Pycnometer)*	4440.5	4440.5	4440.5	4440.5
11. *Weight of Pot + Concrete*	10282	10247	10636.5	10015
12. *Weight of Pot + Concrete + Water to fill*	11931.5	11915	12140	11780
13. *Temp. of Water, ° C*	19.7	19.4	19.3	19.3
14. *Repeat (12)*	11931.5	11914.5	12140	11779.5
15. *Repeat (12)*				
16. *Temp. of Water*	19.8	19.6	19.5	19.5
17. *Density of Water*	.9987	.9987	.9987	.9987
18. Weight of Concrete, (11) — (10)	5841.5	5805.6	6196	5574.5
19. Weight of Water, (14) — (11) or (15) — (11)	1649.5	1667.5	1503.5	1764.5
20. Volume of Water, (19) ÷ (17)	1651.6	1669.7	1505.5	1766.8
21. Volume of Pot	4104	4104	4104	4104
22. Abs. volume of concrete, (21) — (20)	2452.4	2434.3	2598.5	2337.2
23. Bulk volume of concrete, (V) × (18) ÷ (9)	2476.2	2460.1	2626.9	2361.5
24. Volume of Air, (23) — (22)	23.8	25.8	28.4	24.3
25. Per Cent Air, 100 × (24) ÷ (23)	0.96	1.05	1.08	1.03
26. Absolute Specific Gravity of Concrete, (18) ÷ (22)	2.382	2.385	2.384	2.385
27. Bulk Specific Gravity of Concrete, (9) ÷ (V)	2.359	2.360	2.359	2.361

in these tests, and in all others reported in this paper, was very thoroughly puddled into the 0.1 cu. ft. measure to eliminate, in so far as possible, entrapped air that otherwise might be present from careless placing.

Effect of Size and Grading of Aggregate on Air Content. Four tests illustrating the effect of aggregate size and grading on air content of mixtures made of the same cement, and at approximately the same slump and water-cement ratio, are reported in Table 3. The same general type of aggregate grading was used in the first three tests, but the maximum sizes were ⅜ in., ¾ in., and 1½ in., respectively. In the fourth test, the maximum size of aggregate was ⅜ in. with a gap grading. These gradings are shown in Fig. 2A.

Table 3 illustrates the usual method of computing concrete quantities from experimental batches, all the data being secured from the air determinations made as indicated in Table 2. Attention is called (1) to the very considerable reduction in cement factor and air voids as the size of aggregate is increased without materially changing the type of grading, (2) to a marked reduction in air voids by the substitution of a gap grading for the continuous grading in the ⅜ in. max. size.

Effect of Reducing the Cement Factor on Air Content. Table 4 gives the data of 5 concrete mixtures designed with the same quantity and

TABLE 3—DATA OF FOUR CONCRETE MIXTURES WITH DIFFERENT SIZES AND GRADINGS OF AGGREGATE

Max. Sizes of Aggregate	⅜″	¾″	1½″	⅜″
Batch Quantities				
Weight of Cement (lb.)	3.36	2.76	2.43	2.76
Weight of Fine Aggregate (lb.)	9.66	7.28	5.69	3.86
Weight of Coarse Aggregate (lb.)	3.57	5.95	7.54	7.16
Weight of Water (lb.)	1.92	1.60	1.42	1.58
Weight of Batch (lb.)	18.51	17.59	17.08	15.36
Weight of .1 cu. ft. of concrete (lb.)	14.19	14.57	14.79	14.46
Vol. of Batch (cu. ft.)	.1304	.1207	.1155	.1062
Batcher per yard	207.1	223.7	233.8	254.2
Quantities per cu. yd.				
Cement per cu. yd. (lb.)	696	617	568	702
Cement per cu. yd. (sacks)	7.40	6.56	6.04	7.47
Aggregate (lb.)	2740	2960	3093	2801
Total Water per cu. yd. (lb.)	398	358	332	402
Net Water per cu. yd. (lb.)	371	328	301	374
Net Water per cu. yd. (gal.)	44.5	39.4	36.1	44.9
Water-Cement Ratio (gal./sack)	6.01	6.01	5.98	6.01
Air Content, Per Cent	2.8	1.65	1.25	1.3

grading of aggregate to have the same slump, but water-cement ratios varying from 4.5 to 8 gal. per sack. This of course provides a range of mixtures from rich to lean. The aggregate grading is that referred to previously as our regular laboratory grading (Fig. 2B, Curve VI). As in the case of Table 3, all the data are derived from the procedure for determining air content.

TABLE 4—EFFECT OF CEMENT CONTENT

Nominal Gallons per sack	4.5	5.25	6	7	8
Parts by Weight : Cement to Aggregate	1 : 3.26	1 : 4.29	1 : 5.09	1 : 6.30	1 : 7.38
Weight of Cement (lb.)	4.06	3.09	2.60	2.10	1.79
Weight of Aggregate (lb.)	13.23	13.23	13.23	13.23	13.23
Weight of Water (lb.)	1.75	1.58	1.53	1.44	1.41
Weight of Batch (lb.)	19.04	17.90	17.36	16.77	16.43
Weight of .1 c. f. of concrete (lb.)	14.78	14.81	14.75	14.71	14.67
Vol. of Batch (cu. ft.)	.1288	.1209	.1177	.1140	.1120
Batches per cu. yd.	209.6	223.3	229.4	236.8	241.1
Quantities per cu. yd.					
Cement (lb.)	851	690	596	497	432
Aggregate (lb.)	2773	2954	3035	3133	3190
Water (lb.)	367	353	351	341	340
Weight per Cu. Yd. (lb.)	3991	3997	3982	3971	3962
Cement (sacks)	9.05	7.34	6.34	5.29	4.60
Net Water (gal.)	40.7	38.8	38.5	37.2	37.0
Gal. per sack	4.50	5.29	6.07	7.03	8.04
Per Cent Air	1.0	.95	.85	1.05	1.25

The comparatively small variation in the air voids of the above mixtures was quite unexpected, as we had anticipated a gradually increasing air content from rich to lean. This is probably to be explained in part by the fact that no change was made in the aggregate grading, and the aggregate is too fine for the richer mixes. The 4.5 gal.

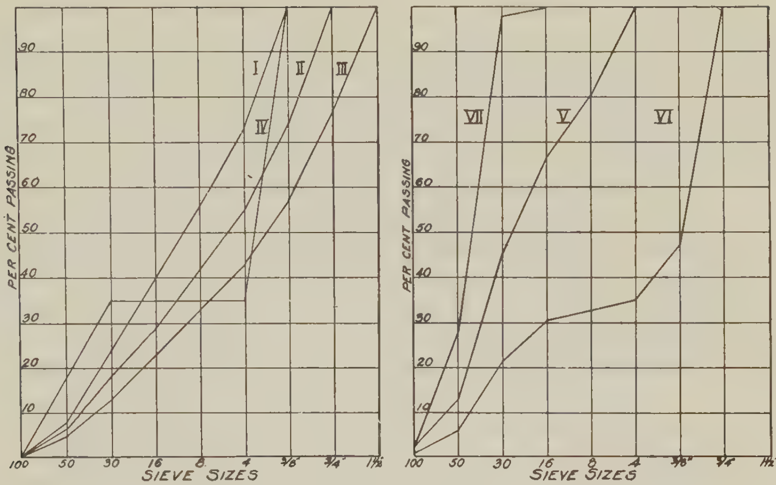

FIG. 2A (LEFT) FIG. 2B (RIGHT)—GRADINGS OF AGGREGATES USED IN TESTS FOR AIR

mix, and to a less noticeable extent, the 5.25 gal. mix, contain an excess of fine mortar in which air is readily entrapped and difficult to remove. Consequently these mixtures, as placed in the 0.1 cu. ft. measure, have slightly higher air content than the 6 gal. mix. But even in the 8 gal. mix, computation of the absolute volumes shows that the paste content is greater than the sand content, and that the mortar content is greater than the coarse aggregate content. Under these conditions the interspace in the sand and in the coarse aggregate would each have to be well above 50 per cent before entrained air could be expected to be present in considerable quantity.

These observations and others suggest that entrained air is primarily a matter of voids or interspace between particles of any group size, which can generally be kept low enough to be filled by the finer portion of the mix with proper attention to grading. Thus, if a concrete mix is not undersanded, the entrained air is more likely to be inherently present in the mortar, and the air content of the concrete will not be much affected by the amount of the coarse aggregate.

Air Content of Plastic Mortars. Reference has been made to the relatively high air content of the standard plastic mortar specified by A. S. T. M. for compression tests. Table 5 presents the results of air determinations on five of these mortars made with different cements in the specified 1:3 proportions of cement to graded Ottawa sand, and with a fixed water-ratio of 0.8. The consistency of these mortars

varies considerably, as would be expected. Mortar No. 6 in Table 5 was made in the same manner, but with Delaware River Sand instead of Graded Ottawa Sand. The quantities of air as determined by the pot test and by computation of absolute volumes of ingedients are given for comparison. Gradings of the sands are given in Fig. 2B, Curve V being the river sand, and VII the Ottawa sand.

TABLE 5—AIR CONTENT OF PLASTIC MORTARS

No.	Cement	Air Content, Observed	Per Cent Computed
1.	Standard Portland	6.45	5.8
2.	Low Aluminate Portland	5.5	4.9
3.	High Early Strength Portland	5.29	4.5
4.	Standard Portland (Made about 1924)	9.05	9.0
5.	Same as No. 1, with .1 per cent water-repellent admixture	13.2	13.4
6.	Same as No. 1, River Sand	3.9	2.6

With reference to the agreement between observed and computed air voids, it will be noted that computed values are smaller except in the mortars of very high air content. From the results obtained it appears that fineness of the cement has a direct bearing on the entrained air, as the first four cements differ appreciably in this respect, No. 4 being decidedly the coarsest, then Nos. 1, 2 and 3, in that order. The remarkable effect of the 0.1 per cent admixture in doubling the already high air content of mortar No. 1, needs to be seen to be appreciated. The consistency reminds one of a cake dough lightened with baking powder. It may be remarked that the generally high air voids in the graded Ottawa sand mortars as compared with those in river sand mortars have lead some investigators to suspect that this sand has a special affinity for air. The authors are now inclined to believe that this is not the case, and that the high air voids are simply the result of an overlean mix, magnified by the lack of suitable grading.

FURTHER SUGGESTIONS IN THE USE OF THE METHOD

The data which have been presented in this paper are limited in extent, but serve to illustrate fairly well the type of results obtainable with a large pycnometer and a balance of suitable capacity and accuracy. Certain points may be mentioned for the benefit of those who may be interested in using the method.

1. Experience with the 5 in. diam. pot indicates that the error in gaging the water level is considerably less than the error in weighing. Hence there should be no inconsistency in using a larger pot, even up to 7 in. diam., if desired. A pot of this size would permit operation with a 25-lb. sample of concrete, containing aggregate up to 2½-in. diam.

2. Weighings accurate to the fourth or fifth significant figure require some care and assurance that the weights are correct. Balance and weights should be calibrated.

3. A small rubber bulb type of syringe is more convenient for gaging the liquid level than a pipette. It is also useful for removing foam and scum. Small amounts of the latter which interfere with a clean liquid surface and accurate gaging can be dispersed by touching the surface of the liquid with the tip of the finger moistened with kerosene.

4. The reduction in volume caused by the reaction between cement and water is more or less appreciable for some time after mixing. It is therefore desirable to make the volume tests not earlier than 10 or 15 minutes after mixing the concrete. The magnitude of the error upon dilution in the pot is probably small, but merits further study.

5. When room-dry aggregates are used in making up the test batches, it should be borne in mind that the absorption of the aggregates may remove a considerable volume of water from the paste. This goes on for some time at a diminishing rate, and any water that may be absorbed between the bulk volume determination and the gaging of the liquid level for the absolute volume determination will appear as so much additional air. It is therefore good practice to allow the aggregate to soak up the mixing water for some time prior to mixing the batch.

6. In filling the 0.1 cu. ft. measure (or any other measure that may be used for the bulk volume determination), a glass plate carefully slid across the top of the very slightly over-filled pot will avoid excess or deficiency, such as may occur from screeding with a straight edge. (See uniformity of weights, Item 9, Table 2).

7. Any considerable temperature change during the procedure will produce errors. The work is best done when all materials and apparatus are kept in a room at nearly constant temperature.

CONCLUDING REMARKS

In conclusion, it may be stated that while the method described herein was developed primarily for the determination of air in plastic mixtures, it has proved very useful for the ordinary laboratory operations on aggregates and other materials. Thus we have found this scheme to be the simplest we have ever used for routine determinations of specific gravity, and the accuracy is such that variations between different samples from the same stock pile can be readily detected if such variations are as great as .01. In fact the apparatus is wel adapted for determinations of absorption and free moisture, and the percentage of silt can be determined without a drying operation if its

specific gravity is known or assumed to be the same as that of the aggregate. Furthermore the specific gravity of cement can be accurately determined by using a large sample in kerosene, the specific gravity of the latter having been first determined with the same apparatus. The apparent increase with time in the specific gravity of cement in water is an interesting phenomenon to follow by this method, and further study of it is desirable for a more accurate measure of the shrinkage in volume which occurs when cement or paste are mixed with water.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Economics of Ready-Mix Versus Job-Mix Concrete*

Report of Committee 610

BY R. L. BERTIN†, AUTHOR CHAIRMAN

Economics, strictly speaking, is a science of the laws of production and distribution of wealth, including all the causes of prosperity and its opposite. For the purpose of this paper, the application of this broad science to the problem at hand is narrowed down to the consideration of the relative financial advantages accruing to a contractor in fulfilling an obligation to furnish a concrete structure by either

(1) Purchasing the concrete delivered at the site, ready for use, designated as ready mixed concrete, or

(2) Purchasing the raw materials processed more or less and fabricating the concrete at the site, designated as job mixed concrete.

The multiplicity of factors affecting the cost of producing concrete is such that it is impossible to give monetary data except for specific cases. This paper, therefore, is presented merely as a guide to assist the contractor in selecting the most advantageous method of supplying his job with concrete of required quality.

Ready mixed concrete, also designated as pre-mixed concrete, is defined in the A. S. T. M. Specifications for Ready Mixed Concrete (C94-35) as mixed concrete delivered at the work ready to use.

Ready mixed concrete is produced by one of two methods:

(a) "Centrally Mixed Concrete" or "Central Plant-Mixed Concrete" where the concrete ingredients are proportioned and mixed in a central plant and then hauled to the job in trucks generally provided with agitators.

*Received by the Institute Secretary, Dec. 11, 1935. Subsequently reviewed and in general approved by the following members of the Committee: R. B. Young, Stanton Walker, G. E. Floyd, J. E. Kalinka. For presentation 32nd Annual Convention, Chicago, Feb. 25-27, 1936.

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(b) "Truck Mixed Concrete" or "Central Plant-Proportioned Truck Mixed Concrete" where the concrete ingredients are proportioned at a central plant and mixed in the conveying trucks while in transit to the job or after arrival. In this case, the necessary water is carried separately in a measuring tank and added when the mixing operation is started.

The sale of ready mixed concrete has in the last 15 years become an important industry, particularly in large centers where there is a demand for this commodity.

Many of the ready mixed concrete manufacturers are members of the National Ready Mixed Concrete Association with headquarters in Washington, the purpose of which is to promote the use of their product and disseminate accurate information concerning its preparation and use.

The engineering division of the association publishes from time to time bulletins designed to furnish information of interest to the users and producers of ready mixed concrete.

During the period of development of this industry, the mechanical means of producing and transporting concrete have been vastly improved and the great majority of plants are now equipped to furnish concrete meeting the most rigid specifications.

The facilities for accurately proportioning the batches, controlling the grading of the aggregates, their moisture content, the water cement ratio, etc., are far more efficient than found on the average job.

This factory made concrete, therefore, may be expected to possess more uniform characteristics than job mixed concrete, which may result in better concrete structures if this uniformity of the original product is not vitiated through segregation resulting from improper handling and placing.

While ready mixed concrete is generally regarded as a product purchased from one engaged in its manufacture solely for profit, there are cases where contractors have set up central mixing or proportioning plants of their own, used purely to supply one or more of their operations.

The purchase of fresh concrete must be based on specific requirements from the standpoint of quality and measurement from the standpoint of equity to both the purchaser and seller.

The quality of fresh concrete may be specified in a number of ways:

(1) by arbitrary proportions;

(2) by cement content per cubic yard;

(3) by water cement ratio;

(4) by compressive resistance at a given age.

Apart from the characteristics fresh concrete must possess in order to produce set concrete of the requisite quality, it must be of a consistency or fluidity when delivered such that it can be placed at a reasonable cost.

The degree of fluidity is not necessarily a fixed index. For instance, in concreting structures composed of a variety of small members crowded with reinforcing bars, thin and high columns or walls, and large slabs, it is often desirable to alter the consistency of the concrete to insure proper compaction and uniformity throughout the work. In high pours, it is desirable to alter the mix slightly to eliminate the accumulation of fines at the top of each pour. Such corrections are difficult if not impossible to make where the control of the mix is remote from the site. It has been observed that, furthermore, the consistency or fluidity of concrete changes with the time of mixing. Long mixing periods tend to stiffen the mass due in part to a grinding action which tends to increase the percentage of fine materials in the mix and perhaps the absorption of an increasing amount of the water by the aggregate and the cement.

Ready mixed concrete cannot readily be altered once received. Transit mix, on the other hand, can be corrected at the job since the transporting equipment is, in reality, a mixer. The only remedy, however, is through the addition of water which under some specifications may vitiate the acceptability of the concrete.

In considering ready mixed concrete for a job, the purchaser should satisfy himself that the dealer is equipped to supply the job at the maximum required rate, without interruption. This is of particular importance as delays in delivery involve serious loss of time on the part of the gangs placing the concrete.

In localities where heavy traffic is encountered between the central plant and the job, resulting irregularities in delivery may be obviated by having an ample number of trucks in transit or providing a reservoir on the job of sufficient capacity to smooth out the gaps between trucks.

There is, however, a disadvantage to this procedure in case of a sudden stoppage in the concreting process on the job due to unforeseen causes, insofar as the concrete in the trucks in transit or in the reserve bin on the job may have to be wasted and may be charged against the purchaser in case the seller is unable to divert the trucks to some other job.

Transit mix does not offer the same disadvantage since the mixing operation can be delayed a much longer time, governed largely by the

moisture content of the aggregate which, if present in sufficient quantity, may spoil the cement.

Where ready mixed concrete is used on a job, a more careful scheduling of pours is necessary than is the case with job mixed concrete and close coordination between the plant and the job during the period of delivery is necessary to minimize the disadvantages described above. In comparing the cost of job mixed and ready mixed concrete, the purchaser must weigh these factors and allow for them lest he be faced with indirect costs which well may upset his direct cost comparison.

In determining the quantity of concrete delivered, some easy method of measuring the mass must be established either directly or by summing up the absolute volumes of the ingredients in a batch.

The verification of quality and quantity involves a considerable amount of testing and supervision which is not within the province of this paper, except insofar as it affects the cost of ready mixed concrete to the user. In some cases, this supervision is conducted by the users own concrete technicians. If not available, the services of a testing laboratory may be secured to verify the quality and quantity of the concrete delivered, much as is done in the case of structural steel.

The Tentative Specifications For Ready Mixed Concrete of the American Concrete Institute (504-31T), the A. S. T. M. Specifications (C94-35), and the literature of the National Ready Mixed Concrete Association, all contain information valuable to the prospective purchaser of ready mixed concrete.

For the purpose of comparing the cost of ready mixed concrete with that of job mixed concrete, it is essential that all costs direct or indirect engendered by the process of combining the ingredients into concrete on the job be established and assembled into a cubic yard unit. Such an analysis involves many factors which are dependent on the nature and magnitude of the job, the costs of raw materials, plant installation, rentals and dismantling, insurance, supervision, etc. While most contractors are fully capable of establishing such costs, it is believed of interest to discuss this phase of the problem.

The various methods of concrete making on the job are:

(a) Receiving the materials in trucks as required, storing the cement on platforms, protected by canvas, and the aggregate in piles from which wheel barrows are filled manually and wheeled to a mixing platform or a power mixer.

This method, the most economical from the standpoint of plant, is used where the rate of concrete supply is small and the total quantity of concrete is too small to absorb economically the cost of more extensive plant equipment. This method is inadequate where accurate control of the concrete proportions is required.

(b) The materials are received in trucks or cars, the cement stored in cement sheds and the bulk aggregate in bins directly from the trucks or conveyors, the

TABLE 1

The total cost of job mixed concrete is made up of the following items:

1. Plant
   - (a) Transportation to and from the job
   - (b) Installation and dismantling
   - (c) Rental
   - (d) Operation
2. Materials
   - (a) Cost of raw materials
   - (b) Labor cost of receiving and storing
   - (c) Handling cost, including measuring and mixing
   - (d) Loss due to bulking of aggregate
3. Incidentals
   - (a) Supervision
   - (b) Testing
   - (c) Insurance
   - (d) Accounting

TABLE 2

The total cost of ready mixed concrete is made up of the following items:

1. Plant (For receiving concrete)
   - (a) Transportation to and from the job
   - (b) Installation and dismantling
   - (c) Rental
   - (d) Operation
2. Concrete
   - (a) Cost of concrete
   - (b) Labor cost of receiving
   - (c) Loss due to waste
   - (d) Loss due to shortage of quantities received.
3. Incidentals
   - (a) Supervision
   - (b) Testing
   - (c) Insurance
   - (d) Accounting

cement being fed from the storage shed to the mixer by wheel barrows or conveyors and the aggregate directly from the bins into measuring equipment of the volumetric, weight or inundation type.

This type of plant is applicable to almost any concrete job of sufficient size to absorb the cost of the plant. In other words, where the labor saved in the handling of the materials is sufficient to offset the cost of the plant and operation. With this type of plant, concrete capable of meeting the most rigid specification can be produced.

In very large operations, suitably located, the cement may be received in bulk, stored in specially constructed bins which are charged by means of cement pumps and fed to the mixer in the correct amount by means of weighing equipment.

In some localities, the fine and coarse aggregate may be purchased in properly proportioned batches which are dumped from trucks directly into the mixer hopper as required.

This method of delivery eliminates the plant charges and labor involved in the storing, handling and measuring of the aggregate on the job. In certain cases, the cement in correct quantity is added to each batch on the truck, thus further reducing the plant and labor charges on the job.

In analyzing the job, the well informed contractor will select the method which yields the most economical production of concrete consistent with the requirements of the job and the specifications.

The cost thus obtained furnishes the basis of comparison between that of job mixed concrete and the price quoted for ready mixed concrete by the dealers.

Where the use of plant is involved in these calculations, there may be a considerable difference of opinion between the contractors as to the value chargeable to the job. Many indirect expenditures are frequently overlooked with the result that the cost of concrete per yard, based thereon, may be fictitious. With ready mixed concrete, purchased at so much per yard the elements of uncertainty are saddled by the dealer and the contractor knows exactly what his concrete will cost him, except that this price may be modified by excessive labor costs on the job due to unsatisfactory delivery or loss of concrete as pointed out elsewhere in this paper.

In Table 1 and 2, the various items which go to make up the cost of concrete on jobs both for job mixed and ready mixed concrete are given.

Table 3 is a list of plant items which a contractor must price according to the local conditions, selecting from the list such items as are required for the type of plant he chooses to use for a particular job.

It is recognized that certain cost items such as fuel or power, supervision, operators, etc. are difficult to isolate from the rest of the job plant cost but the value of these items compared to the total cost is small and can be approximated without much error.

Where the nature of the job is such that the ready mixed concrete can be discharged directly from the trucks into the forms, the equipment or plant required to distribute job mixed concrete may be saved. On jobs where space is not available for the storing of concrete materials or erecting a mixing plant, ready mixed concrete is of course the only alternative.

In some cases, contractors have found it more advantageous from the standpoint of cost to construct their own central mixing plant and feed one or more jobs from it than to purchase ready mixed concrete from a dealer whose plant may be too far away from the job or whose sales prices may be too high to compete with the cost to the contractor of manufacturing and transporting the concrete he requires.

Aside from the true difference in cost between job mixed and ready mixed concrete a contractor may purchase more expensive ready mixed

TABLE 3—JOB PLANT

	Material	Transportation to and From Job	Erection	Rental	Dismantling
CEMENT					
Storage at Site					
(a) Platform and canvas					
(b) Shed					
(c) Tank and pump					
Handling Equipment					
(a) Conveyors					
(b) Bag bundling machine					
Measuring Equipment					
(a) Weighing machine					
AGGREGATE					
Storage at Site					
(a) Platforms					
(b) Bins					
Handling Equipment					
(a) Conveyors					
(b) Derricks					
(c) Carryalls					
Measuring Equipment					
(a) Volume batchers					
(b) Weighing batchers					
(c) Inundators					
Heating Equipment					
(a) Steam coils					
WATER					
Conveying and Storage					
(a) Piping or hose					
(b) Pump					
(c) Tank					
Measuring Equipment					
(a) Volumetric tank					
Heating Equipment					
(a) Steam jet or coils					
MIXING					
(a) Platforms for hand mixing					
(b) Power Mixers					
POWER EQUIPMENT TO OPERATE THE ABOVE LISTED MACHINERY					
(a) Electric Motors					
(b) Gasoline Engines					
(c) Steam Engines					
(d) Boilers					
(e) Wiring					

concrete from a dealer because it reduces the initial outlay of money for plant equipment and the credit extended him reduces his weekly pay roll, thus reducing the amount of cash necessary to carry on his contracts.

On jobs of the type of a multi story reinforced concrete building, it has sometimes been found economical to purchase ready mixed concrete for the installation of the foundations when small daily quantities of concrete are required and the erection of the main mixing plant would interfere with the progress of the job at that stage. After the floors and roof have been placed from a job mixing plant, resorting again to the use of ready mixed concrete for the pouring of odds and ends such as stairs, pent houses, concrete sills, etc., which generally

proceed at a slow rate with the other trades and at which time the main plant would interfere with the finishing trades.

As stated at the beginning of this paper, it is impossible to give specific costs of either ready mixed or job mixed concrete. The costs published in the *Engineering News Record* indicate a range from \$5.45 to \$8.25 per yard for ready mixed concrete between different parts of the country. The range in cost of job mixed concrete for the great variety of job conditions, labor rates, material costs, etc., for different parts of the country, far exceeds that given for ready mixed concrete.

This paper is presented as a general exposition of the application or use of both ready mixed and job mixed concrete from the standpoint of the contractor. It is hoped that it will be discussed freely and much additional information from actual experience furnished thus enhancing its value to those interested.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Aggregate Production for Grand Coulee Dam*

BY GORDON F. DODGE†

As a prelude to the subject under discussion, it is possibly desirable to give a brief description of the surroundings of the Grand Coulee Dam on the Columbia River in north central Washington.

The country rock of this district is granite which, during the earlier geologic periods preceding the glacial age, was covered by seven or more successive lava flows, all separated by considerable periods of time as evidenced by fossil trees as large as seven feet in diameter in at least one of the lower partings.

During the glacial age at least two, possibly three successive glacial movements occurred, sweeping down from Canada and forming ice and debris dams across the river. The water thus impounded found an outlet to the south into the Snake river valley and, during the times it was thus diverted, cut enormous channels from one to several miles wide and several hundred feet deep through the overlying lava rock. One of these channels named Grand Coulee is about 50 miles long. At about its mid length is an ancient waterfall (dry before Niagara was born) which is more than twice as high as Niagara and which carried many times as much water. As the glaciers receded, the dams which had been formed were worn away and the river resumed its old bed leaving the topography as it now exists, with the bed of the temporary channel about 450 ft. above present river level.

This Grand Coulee has given its name to the dam which is now being constructed across the Columbia a short distance down stream from its junction with the temporary channel.

As a construction project this dam is notable in that when completed it will undoubtedly be the largest mass of concrete ever placed. It is to be of the straight gravity type with a length from end to end of approximately 4,300 ft., a maximum height from lowest bed rock of approximately 550 ft., a maximum width at the base of approximately

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500 ft., and will raise the low water level about 370 ft., forming a lake about 151 miles long, extending to the Canadian border.

When completed this dam will contain more than 11,000,000 cu. yds. of concrete and finding a suitable source of aggregates for such a mass of concrete was no small task.

However, nature seems to have had a long foresight and through the agency of the glaciers ground up and laid down a large bed of sand and gravel but a mile and a half from the east abutment of the dam. This deposit while not ideal in its composition is of sufficient quantity and contains materials of acceptable quality after processing.

Because of its lava cap and granite base rock origin, the deposit is a mixture of granite and basalt with the basalt portion being sufficient to give the resulting sand and gravel a very dark color. The high percentage of basalt has the beneficial effect of reducing the mica content in the sand to an acceptable percentage, particularly as the mica has been ground very fine.

This deposit while washed and spread by the waters of the glaciers is still quite different from the usual river bar. Its top is at an average of about 950 ft. above the present river level and, unlike the usual river deposit, where the coarsest material is always upstream, the order of deposition is more or less reversed, with coarser materials at points farther along the path of movement, as is frequently the case with glacial deposits.

With all of its beneficial foresight, however, Nature played one scurvy trick in that she kept her mills working too long and hard and made an excessive amount of sand, some of it exceedingly fine, as will be seen by an examination of the screen analyses of Test Pits No. 6 and 7 reproduced in Tables 1 and 2. To compensate for this, she refrained from laying down any clay above a certain zone, at least insofar as the few test pits have shown.

For the purpose of making a study of processing plant requirements, the test data available were not as complete as might have been desired, especially in view of the large area and depth of deposit that must necessarily be mined. Outside of some surface trenches on the slopes, only the 4 ft. square Test Pits 1, 4, 5, 6 and 7 shown on the log sheet in Fig. 1, together with screen analyses of these pits, were available at the start. It was obvious that suitable material mineable without prohibitive waste did not exist below El. 1600. The test pits together with surface markings made it almost a certainty that an indicated sand layer just below El. 1800 extended throughout the deposit. This layer of sand is extremely fine, in fact too fine to be of any use

TABLE 1—NO. 6 TEST PIT ANALYSIS

Elevation	Depth	Gravel cu. ft.	+ 3"	+ 1½"	+ ¾"	+ ⅜"	+ No. 4	Sand	Sand cu. ft.	+ No. 8	+ No. 14	+ No. 28	+ No. 48	+ No. 100	− No. 100	Fineness Modulus
			%	%	%	%	%	%	%	%	%	%	%	%	%	
Top 1882 to 1868.4	13.6	84	47.7	19.0	19.0	9.5	4.8	61.1	132	7	10	9	12	18	44	1.44
1859.5	8.9	53	37.7	22.6	15.1	22.6	2.0	36.9	31	14	19	18	18	15	16	2.51
1857.0	2.5	21	19.1	19.1	38.2	19.1	4.5	52.3	23	10	16	22	21	16	15	2.38
1855.0	2.0	29	55.2	13.8	13.8	13.8	3.4	34.1	15	10	17	21	21	15	16	2.38
1853.5	1.5	120	40.0	26.6	16.7	13.3	3.4	42.3	88	14	31	36	15	3	1.0	3.35
1846.0	7.5	126	44.5	22.2	15.9	12.7	4.7	38.3	78	19	32	31	11	3	4.0	3.41
1838.0	8.0	3	0	0	0	0	10.0	99.0	273	2	3	18	50	22	5.0	1.98
1824.0	14.0	27	0	14.8	29.6	44.5	11.1	92.7	337	5	11	24	31	19	10	2.22
1807.0	17	44	18.2	9.1	18.2	36.4	18.1	80.4	180	16	20	20	13	16	15	2.62
1796.0	11	0	0	0	0	0	0	100	236	1	1	4	3	8	83	0.35
1785.0	11	0	0	0	0	0	0	100	108	0	1	0	0	2	97	0.06
1774.5	10.5	10	40.0	40.0	0	0	20.0	94.8	182	1	2	6	13	13	65	0.70
1767.5	7.5	47	42.5	17.0	17.0	12.0	6.5	57.0	65	20	34	29	11	2	4	3.47
1763.0	4.0	46	34.8	26.1	17.3	17.3	4.5	54.0	54	12	30	37	15	3	3	3.24
1758.0	5.0	79	40.5	25.3	20.2	10.1	3.9	56.4	61	10	23	44	19	2	1	3.19
1753.0	5.0	76	42.1	21.1	15.8	15.8	5.2	44.1	60	9	16	27	22	13	13	2.47

TABLE 2—NO. 7 TEST PIT ANALYSIS

Elevation	Depth	+ 3"	+ 1½"	+ ¾"	+ No. 4	− No. 4	+ No. 8	+ No. 14	+ No. 28	+ No. 48	+ No. 100	− No. 100	Fineness Modulus
		%	%	%	%	%	%	%	%	%	%	%	
El. 1920 to 1896	24	0	0	0	0	100	0	1	10	47	35	7	1.63
to 1884	12	0	0	0	1	99	2	7	23	26	20	22	1.79
to 1876	8	2	7	7	9	75	11	22	39	23	4	1	3.10
to 1870	6	23	14	14	14	34	31	39	25	4	1	0	3.95
to 1860.5	9.5	40	16	12	12	21	29	37	25	6	2	1	3.82
to 1853	7.5	19	16	7	4	54	17	37	35	10	1	0	3.59
to 1846.5	6.5	26	21	15	10	29	19	32	32	12	4	1	3.47
to 1838	8.5	6	10	6	11	66	8	16	33	33	8	2	2.77
to 1825.5	12.5	2	3	7	9	79	10	16	26	31	14	3	2.68
to 1818	7.5	11	11	13	16	48	22	18	24	24	9	3	3.11
to 1811	7	10	13	13	10	54	10	17	29	31	11	2	2.78
to 1800	11	14	10	10	13	53	15	17	23	27	14	4	2.80
to 1795	5	14	6	6	14	60	16	14	9	6	6	49	1.81
to 1786.5	9.5	0	0	0	0	100	0	0	1	1	15	83	.20
to 1780	6.5	0	0	0	0	100	0	0	1	0	6	93	.09
to 1772	8	0	0	0	0	100	0	2	10	23	19	46	1.03
to 1768	4	32	12	9	10	37	28	30	18	9	7	8	3.39
to 1763	5	20	10	7	7	55	19	33	26	15	4	3	3.39
to 1756.5	6.5	33	18	12	8	29	12	39	35	11	2	1	3.45

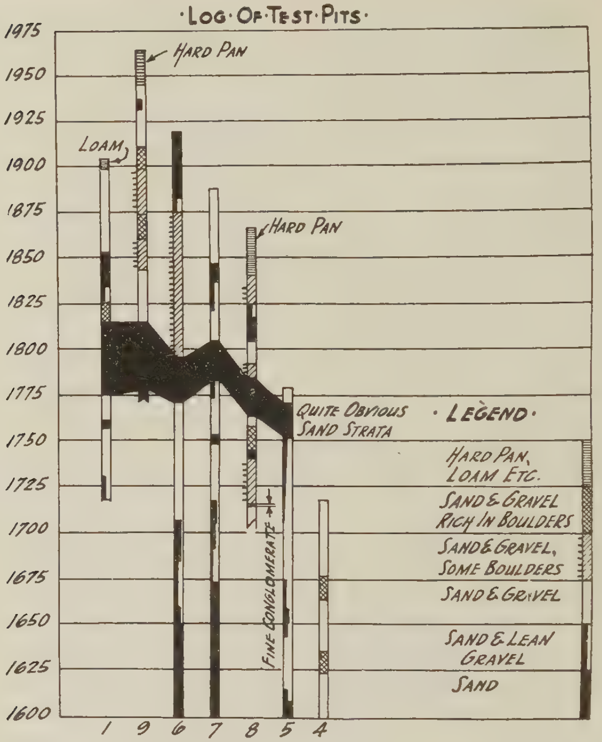


FIG. 1

whatever. Reference to the analysis tables shows fineness moduli as low as .09 and .06, but this does not tell the whole story. As much as 44 per cent of minus 200 mesh was found by further testing of selected samples. It was, however, still sand, not silt.

In view of the ultimate high dam requirements with a possible shortage of needed material lower down in the deposit, it was decided with the accord of the Reclamation engineers that a greater area than originally intended should be opened up for the present contract covering about 4,000,000 cu. yds. of concrete, so that all present requirements could be obtained without going below the fine sand layer. This then left Pits 1, 6 and 7 as the principal early source of information with Pits 8 and 9 following later.

The Reclamation Bureau engineers set a tentative ratio of 2.6 of gravel to one of sand as the desired concrete mix and a grading of sand with upper and lower limits as shown on the accompanying sand curve.

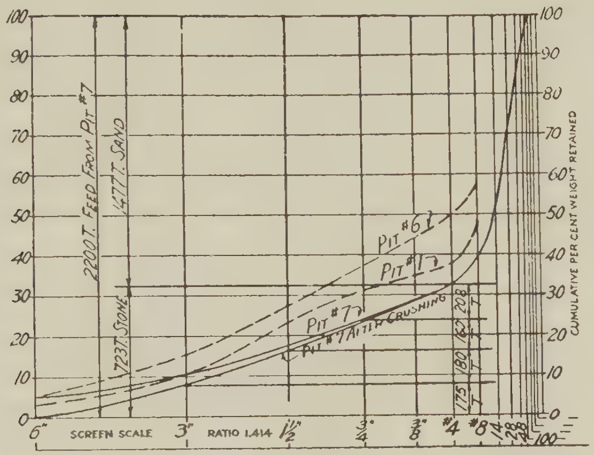


FIG. 2

A study of the test pit data available showed the enormous excess of sand in the deposit and indicated the necessity of providing large waste capacity. It also indicated that the sand screening capacity would be the critical point in the screening plant and would set the screening area required. A vital element in sand screening rates is the moisture content in the sand and, as it was obvious that an absolutely dry feed could not be depended upon at all times, there was nothing to do but make it a wet plant throughout with a vertical flow sheet and an ample supply of water. The surrounding topography further favored that arrangement of screens as all material could be handled with a minimum of lifting through the process.

Plotting of test curves from the available test pit data (Fig. 2) indicated that Pit 7 should probably be about the worst to be expected in the matter of sand tonnage to be handled and that pit was therefore used as a basis of screen plant design. Maximum concrete pouring schedules (later increased) called for an average production of about 740 tons per hour of finished aggregates. To cover variations, an output of 1000 tons per hour was adopted, sufficient for continuous supply to seven of the eight proposed 4 cu. yd. mixers. With the proposed mix ratio of 2.6 to 1.0 this calls for a finished output of 723 tons of gravel and 277 tons of sand per hour. To secure the desired 723 tons per hour of gravel, under average conditions, a plant feed of 2200 tons per hour was indicated, and this gave a sand feed of 1477 tons per hour. However, as sand and gravel deposits are never uniform, a maximum plant feed of 2500 tons per hour with 1777 tons per hour

of sand was adopted to cover variations, and sand screening areas were set accordingly.

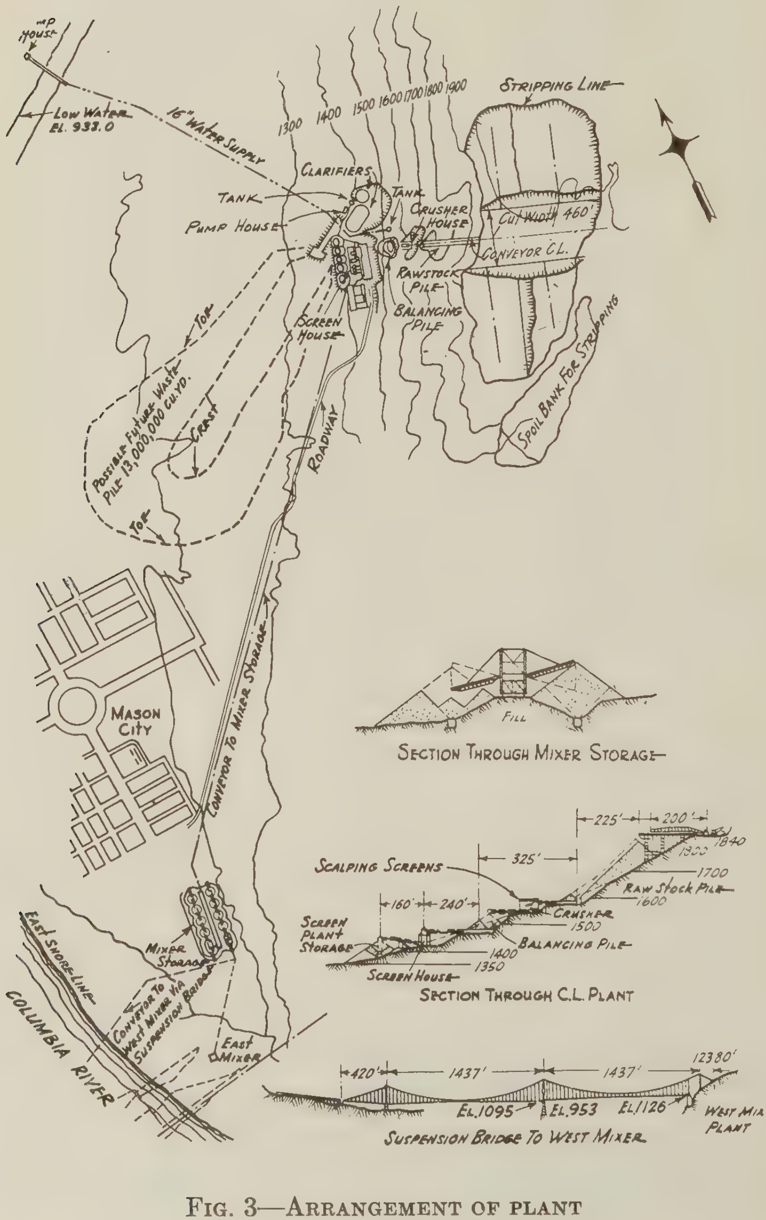


FIG. 3—ARRANGEMENT OF PLANT

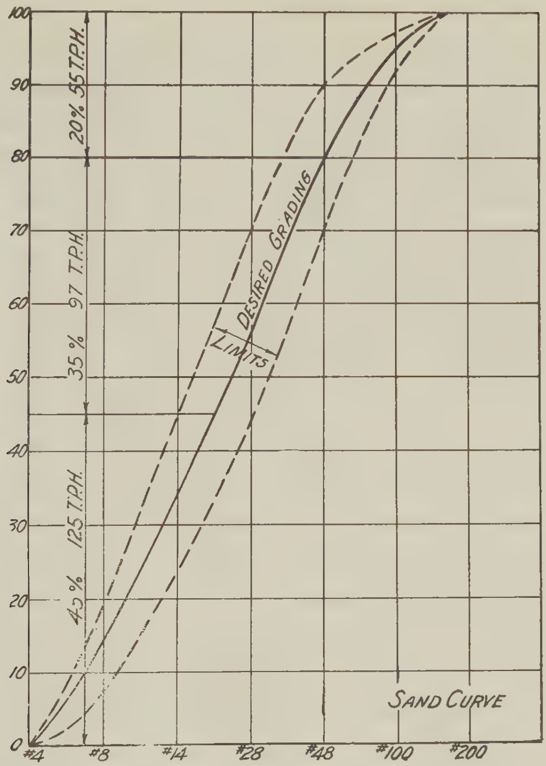


FIG. 4

Due to the steep slope of the face of the deposit (as much as 1½ to 1), no suitably level screening plant site was available except at a point about 475 ft. below the average top of the deposit, and advantage was taken of this site (see cross section of screening plant—Fig. 3) to introduce a large raw stock pile ahead of the crusher station and a fairly large balancing pile between the crusher station and the screening plant, both of which should have an appreciable averaging effect upon the material before it reaches the screens. Also in order to average out the material as much as possible in mining, the taking of two cuts of 40 to 45 ft. depth each was adopted for removing the 80 to 90 ft. depth of material lying above the fine sand layer.

The specifications upon which the contract is based, calls for four sizes of gravel, 6 x 3 in.; 3 x 1½ in.; 1½ x ¾ in.; ¾ in. x 4 mesh, and sand of a fineness modulus of 2.5 to 3.0 and with a grading as shown on the sand curve, Fig. 4. It further states that insofar as is consistent

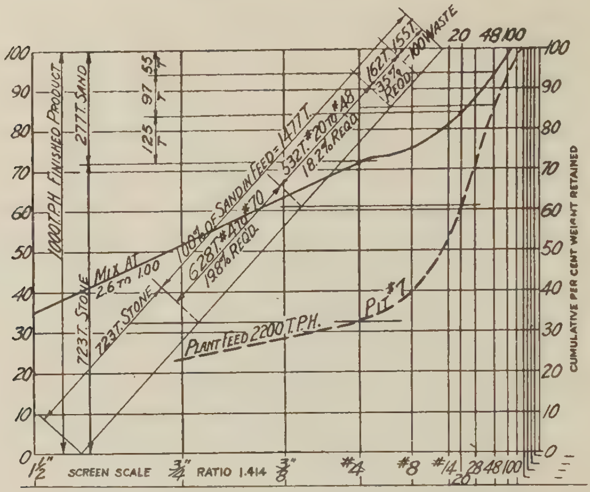


FIG. 5

with the making of good concrete, the full run of the pit production shall be used.

For the gravel portion of the mix this specification introduced no difficulties. It required simply the crushing of oversize above the 6 in. maximum specified, as the gravel portion of the available material is fairly well graded. In case of the sand, however, the problem was more difficult because, along with the large excess volume, it is of a fineness modulus that is on the average too high, as can readily be observed from a study of the test pit data.

To correct this more of the coarse elements have to be discarded; and the simplest way to accomplish that purpose is to split into several fractions and then blend the retained portions in the proper proportions. It was therefore decided to produce three fractions, 4 to 20 mesh, 20 to 48 mesh and 48 to 100 mesh. The sand curve shows approximately what tonnages would be required of each of these fractions.

Fig. 5 shows how these tonnages are applied to the theoretical average plant feed of 2200 tons per hour to determine what portion of the total sand must be taken to the classifiers. The diagonal proportional line shows 628 tons of 4 to 20 mesh whereas 125 tons or 19.8 per cent is required; 532 tons of 20 to 48 mesh with but 97 tons or 18.2 per cent required; 162 tons of 48 to 100 mesh with 55 tons or 35 per cent required; 155 tons of minus 100 mesh which will have largely been eliminated in a previous step of the process, namely, the dewaterers. More about these later.

From the preceding it is seen that to obtain 55 tons of 48 to 100 mesh it is necessary to use a classifier feed of 35 per cent of the total sand output, or about 515 tons per hour. To cover variations a maximum classifier feed of 600 tons per hour was adopted. Obviously, with this tonnage of classifier feed, an excess of the two coarser fractions will be produced and must be disposed of. Classifier discharge chutes were therefore designed to split production in 25 per cent increments and to send any increment either to the stock pile or to the waste system. In each stock pile the material is permitted to drain, and then is moved over a tunnel in which are a belt conveyor and a number of variable speed feeders by means of which the correct proportions of each fraction may be placed upon the conveyor, the actual analyses of the fractions having been determined at the classifiers as they were produced. To insure thorough blending and eliminate all possibility of balling of the fine fraction, the proportioned sand is put through regular foundry sand aerators on its way to the stock pile. The combined fractions are further blended during placement in the stock pile by an automatic belt conveyor tripper that continually runs back and forth the length of the stock pile as the sand is delivered. The effectiveness of this control of the sand grading is shown by operating tests giving moduli running regularly between 2.75 and 2.80.

The sand dewaterers that have previously been mentioned are used for the purposes principally of draining the sand which comes from the screens in a flood of water, putting it into a condition that can be handled on belt conveyors and saving the water for reuse after clarification. A secondary use is the elimination of the majority of the minus 100 mesh material which overflows with the wash water into the clarifiers, where it is settled out and drawn off to waste as a sludge underflow.

Clarification of wash water is not a frequently used expedient where an ample supply of clean water is available, as is the case at Grand Coulee. Its adoption here was determined not by a shortage of water, but by the high pumping head (about 610 ft. static, 670 ft. total dynamic) and the length of line required (about 3400 ft.). A further consideration was that discharge of a possible 20,000 gal. per minute of dirty water into the river might be the cause of complaint. A study was made as to the possibility of utilizing the wash water as a means of disposing of the waste sand, but in addition to having the river contaminated through the natural drainage channel into the river, upstream from the desired pump house location, insufficient sand disposal area was available even for this first contract without a large labor expense in restraining its spread. In fact, comparative estimates in-

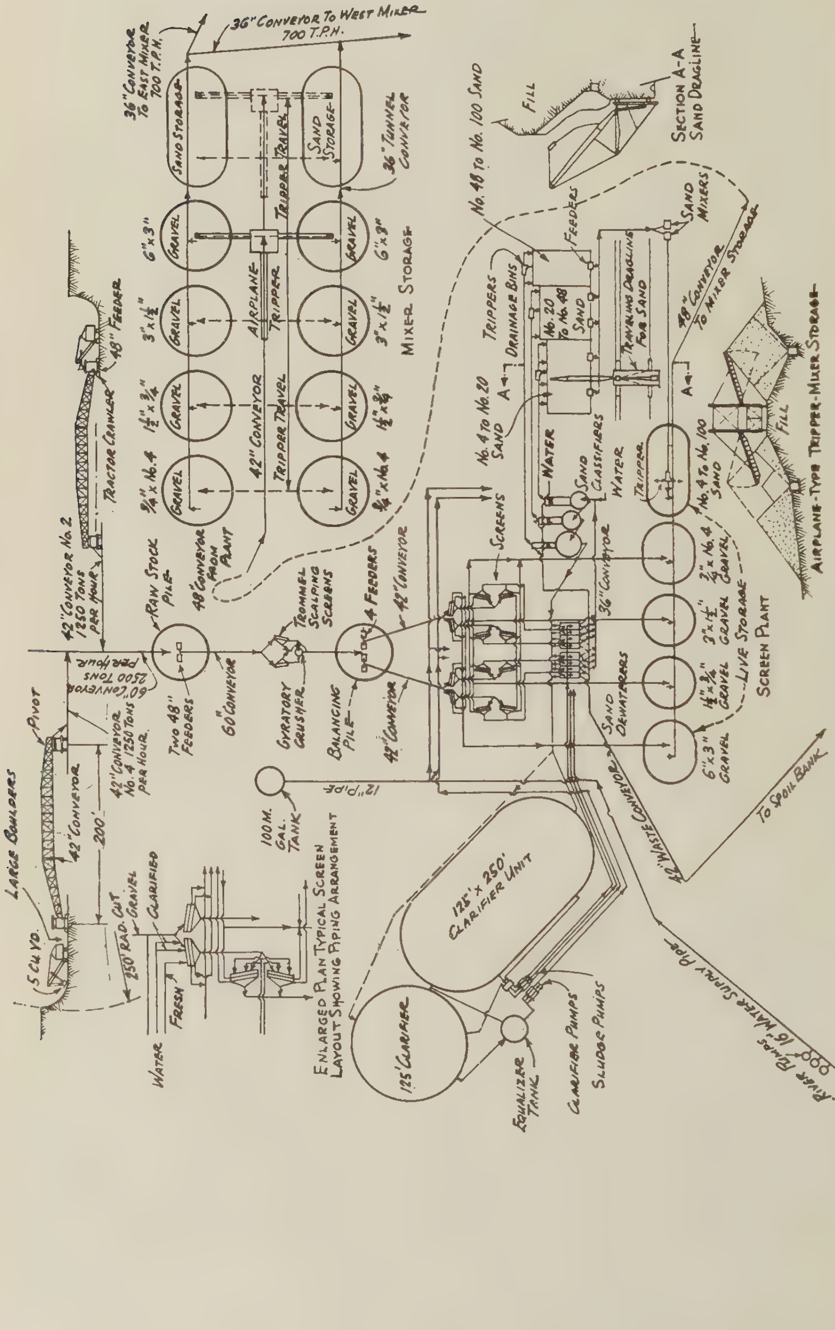


FIG. 6—FLOW SHEET OF MATERIAL HANDLING

dicated that a worthwhile final economy in plant cost and operation could be attained by the use of conveyors as a means of waste disposal, with the added advantage that the waste area available would, with that method, be sufficient for not only this contract but for the total construction project.

Having discussed the principal technical and mechanical problems that had to be solved, a description of the plant provided may be of interest. This is illustrated by the general plant layout in Fig. 3, and the flow sheet in Fig. 6.

Briefly the material is to be mined by two 42-in. radial boom-conveyors, each 200 ft. long. Each boom-conveyor is carried by a truss that is movably and pivotally supported at the discharge end over one of the extensible field conveyors. At the receiving end they are provided with hoppers and feeders all pivotally supported upon a frame carried by two crawler treads. Each crawler tread is independently driven by its own motor so that full flexibility of motion is obtainable for the hopper in any direction in following the shovel. As there is a small percentage of boulders too large for handling on the belt conveyors, each hopper is equipped with a self dumping grizzly for their rejection.

In normal operation the shovel cuts on a semi-circular face with the hopper following closely. After completion of each swing the radial boom-conveyor advances by the depth of the cut and a reverse swing is made. As needed, or after an advance of 160 to 170 ft., a new section of corresponding length is spliced into the extensible field conveyor and the advance continued.

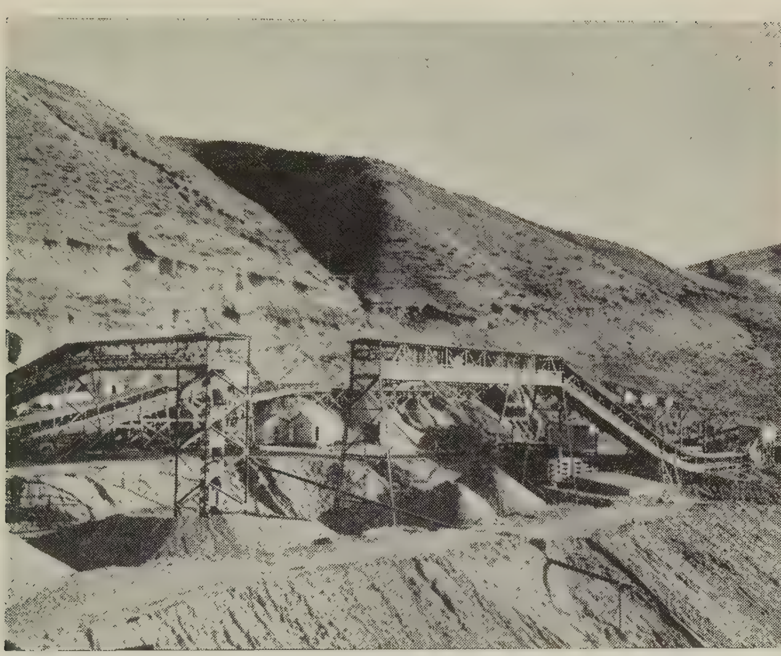
The materials thus dug and delivered to the main field conveyor fall from the end of the trestle into the raw stock pile. From the raw stock pile it is fed and carried by a conveyor to the crusher house where two 72 in. x 22 ft. trommel screens reject the plus 6 in. material into a 20-in. gyratory crusher set to reduce all to minus 6 in. and thus avoid a recirculating load. The throughs of the screens and the crusher product join on an outgoing belt conveyor and are delivered into the balancing pile ready for processing.

From the balancing pile, four feeders and two conveyors deliver to the upper floor of the screen house where a sufficient supply of clarified water is introduced to guarantee a fluid mass on the first screens, which are four in number with 3 in. square openings in the top deck and 1½ in. square openings in the bottom deck. Two sets of spray nozzles supply additional clarified wash water and a third set supplies fresh rinse water to each screen. A hopper under each screen catches

GENERAL VIEW OF PLANT (SEE OPPOSITE PAGE)

the water and the minus 1½ in. material and splits it onto two screens on the next floor down, making eight screens for that stage, each having a ¾ in. square mesh top deck and a No. 4 (square) mesh bottom deck. Spray arrangement is the same on these screens as on those above. Each size of finished gravel from the four upper and the eight lower screens is delivered through "stone ladder" chutes to a system of collecting conveyors on the floor next below the fine screen floor, and from there the different sizes are carried by conveyors to their respective piles over the main storage tunnel. Here also "stone ladders" are used to reduce breakage of finished materials, the piles being some 50 ft. high.

Hoppers below the fine screens catch the sand and water and four steel launders (one for each pair of screens) deliver them to the sand dewaterers. These are simply enormous sand drags (eight in number) in concrete tanks. Two drive units are used, one each for a group of four machines, but clutches permit disconnecting adjacent pairs so that the dewaterers are actually grouped the same as the screens and sand launders.



GENERAL VIEW OF PLANT (SEE OPPOSITE PAGE)

The fine material and water overflow into a wooden launder leading to the clarifiers while the dewatered sand is dragged up and discharged to two belt conveyors, one going to the waste dump and the other to the classifiers. Amounts of sand going to either destination are controlled by four flop gates in the discharge chute of each cell of the dewaterers so that even under maximum sand conditions the diversion increments are 50 tons per hour or less.

Sand to the classifiers is delivered with the necessary water to the first classifier unit where minus 20 mesh is overflowed to the second unit. The second unit overflows minus 48 mesh to the third unit and that overflows the water and undesired minus 100 mesh into the launder going to the clarifiers. The further treatment and handling of the sand into the plant stock pile has already been outlined, as has also the waste sludge from the clarifiers.

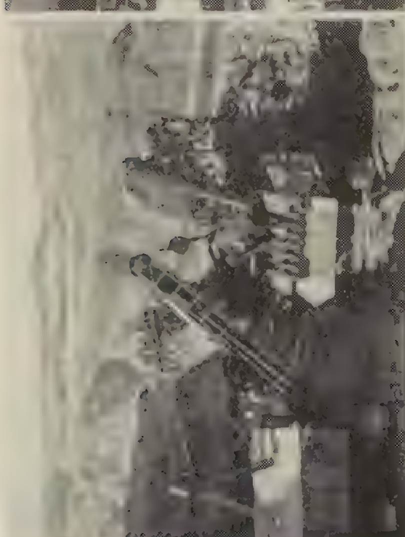
Due to the uncertain, or rather the unknown, factors in the pit, provision has been made to supply water in excess of the old rule of thumb of "a gallon per minute per yard per day." This applies to pump foundations and pipe lines only as pumping equipment has not all been installed. One river pump and one clarified water pump have been left out for later installation if and when needed.

FIG. 8 (UPPER LEFT)—RADIAL BOOM IN PIT

FIG. 9 (LOWER LEFT)—ELECTRIC VIBRATING FEEDER UNDER RAW STOCK PILE

FIG. 10 (UPPER RIGHT)—SCREEN FLOOR

FIG. 11 (LOWER RIGHT)—SAND DEWATERERS



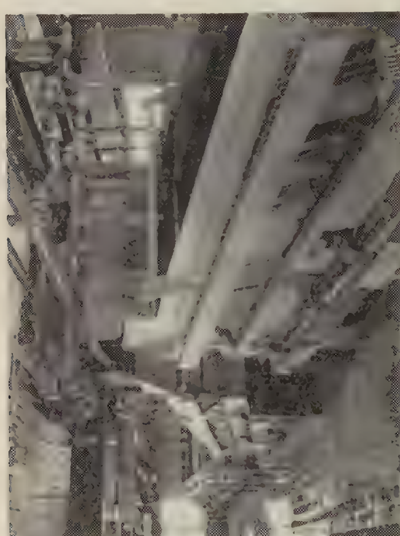


FIG. 12 (UPPER LEFT)—WASTE SAND STACKER

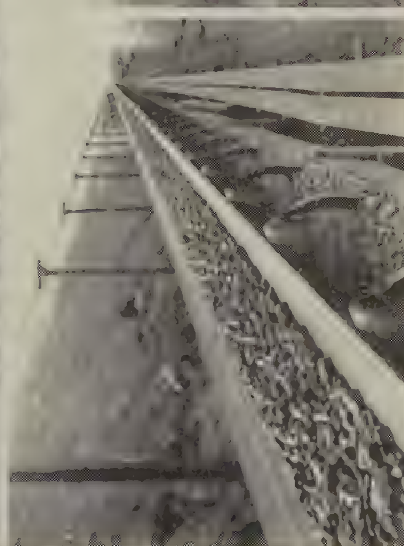


FIG. 13 (LOWER LEFT)—PART OF 1¼ MILE CONVEYOR TO MIXER STORAGE

FIG. 14 (UPPER RIGHT)—AIRPLANE TRIPPER

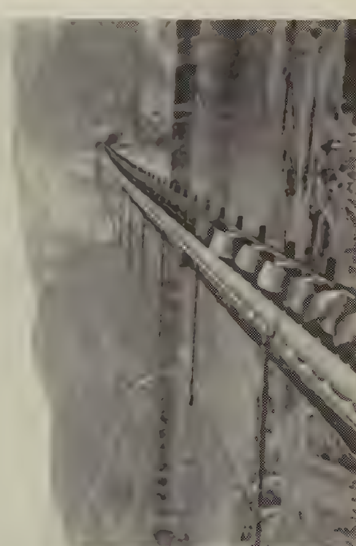


FIG. 15 (LOWER RIGHT)—SUSPENSION BRIDGE FOR CONVEYOR TO "WEST MIX"—3,300 FT. LONG

Due to the considerable moisture carried in the sand that will be wasted, loss of water was allowed for upon the basis of 25 per cent, but clarifier pumps were based upon a possible saving of 90 per cent. From the stock piles at the plant a conveyor system more than 6,000 ft. long transports separately the various sizes of gravel and the blended sand to the main storage piles where about 3½ days' maximum demand of material may be held ready for transport by conveyors to the mixing plant bins.

All storage is on the ground over tunnels, but while stone ladders were used at the plant to prevent breakage, the wings of the aeroplane tripper at the main storage are hinged to permit adjustment to the height of piles and minimize fall of finished gravel.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Concrete by Pump and Pipeline*

BY CHARLES F. BALL†

INTRODUCTION

The system of placing concrete by pump and pipe line known as Pumpcrete was first introduced in the United States late in 1932. In the last two years two million cubic yards of concrete have been pumped in this country alone, and this method has been successfully employed for such a wide variety of construction work that it is no longer regarded as a fantastic notion, but rather, is now recognized as a standard way of delivering concrete. Undoubtedly, a better understanding of just what happens, and a consideration of the factors that make for or against successful pumping of concrete will promote more efficient application on the part of the contractor, and frequently, more economical design on the part of the engineer.

It is the purpose of this paper to discuss the essentials of the pump itself, and associated equipment, and enumerate some of the phenomena of concrete actions in connection with these devices. A further study will be made of some of the concrete mixtures that have been used, (both good and bad from the standpoint of pumping,) in the hope of establishing some helpful generalities.

The Pumcrete system can be defined as the method of transporting fresh (plastic) portland cement concrete through pipe lines by means of direct acting pumps.

In practice, mixed concrete is supplied to the hopper of the pump, and then much in the same manner as with an ordinary fluid, successive charges are drawn into a working chamber (cylinder) and these charges are forthwith expelled into the pipe line. Each charge as it is forced into the line pushes all of the concrete in the line forward and, as concrete is relatively incompressible, a nearly equivalent amount is ejected from the end of the pipe. The pipe line is entirely filled at all times except when first starting.

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CONCRETE A DIFFICULT MATERIAL TO HANDLE

Of the commonly used materials, probably concrete is the most difficult to transport. There are several reasons for this, of which the following are of greatest importance:

1. It is a material that in a relatively short time changes from a semi-fluid to a solid, and there is no known way of thereafter reversing the change.

2. It is a semi-fluid only insofar as the mass as a whole is concerned, for the internal structure consists principally of solid pieces of varying sizes, the ratio of solids to liquids being in the neighborhood of 10 or 15 to 1 by weight. The larger pieces, which form the coarse aggregate, may be gravel, or crushed stone, or slag, or manufactured compositions. The fine aggregate is usually sand, and contains pieces as large as 3/16 in. or 1/4 in. in maximum dimensions, and as small as particles passing a 100 mesh screen or even "powder fine." The coarse and fine aggregate in desired proportions are combined with a binder of cement paste, consisting of cement and water in varying ratios. *It is of the utmost importance to the quality of the structure being placed that the proportions of large pieces, small pieces and cement paste be uniform throughout the entire mass,* and unless the material handling system delivers the concrete in a uniform condition of mixture throughout, it has failed almost as signally as though part of the material handled had been wasted on the way.

3. While the concrete as a mass is semi-fluid in its characteristics, it resists doing anything except staying where it is, or moving in a straight line, preferably down hill. Very dry concrete has a sliding angle of 35 degrees or more. Concrete containing large pieces of coarse aggregate tends to bridge or "log-jam," and this is so pronounced with the drier mixtures that it is difficult to get concrete to drop out of a hopper or bucket, even though a very large opening.

4. Because of the sand content and sometimes because of the sharp edges of the coarse aggregate pieces, concrete is highly abrasive and this characteristic is obviously of great consequence in either chuting or handling by pipe line.

DESCRIPTION AND EXPLANATION

The concrete pump or Pumpcrete machine is a heavy duty, single acting horizontal piston pump involving, as would be expected, several unusual details of construction. Except for extreme ruggedness, some of the constructions are very like a heavy duty water pump of the piston type. A conventional crankshaft and connecting rod design is used. The cylinder is disposed horizontally with the outlet connection directly in line with the cylinder.

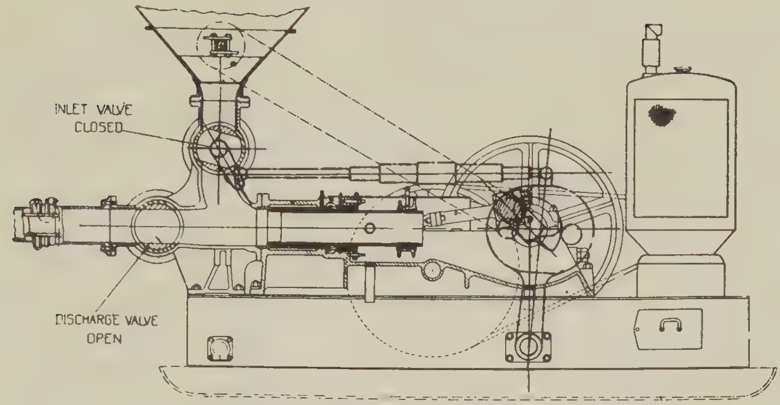

FIG. 1—THE CONCRETE PUMP

A supply hopper is mounted above the cylinder head chamber. An inlet valve of very unusual design is positioned between the hopper and the chamber. An outlet valve of similar type is in the outlet passage. The inlet and outlet valves are glorified plug cocks, and they are mechanically opened and closed in timed relation to the movements of the piston. Their operating mechanism comprises very large double acting cams mounted on the crankshaft which cooperate with rollers carried by oscillating arms. The oscillating arms are connected to the valve plug levers by valve rods which incorporate a spring relief. All of these parts and their relative positions can be identified in Fig. 1.

The cycle of operation is as follows: The inlet valve is opened at the beginning of a suction stroke, and a charge of concrete from the hopper is drawn down through the inlet valve into the cylinder as the piston retreats. At the end of the suction stroke the inlet valve closes, and the outlet valve opens, and the charge of concrete in the cylinder is pushed forward by the advancing piston. This forces concrete out of the pump chamber through the open outlet valve into the connecting pipe line. At the end of the pressure stroke, the outlet valve closes and the inlet valve is opened and a fresh charge of concrete is drawn into the cylinder on the next suction stroke. The operation is obviously two cycle and one charge of concrete is handled for each stroke of the piston.

Except for the mechanical opening and closing of the valves which is not usual with water pumps, the action is substantially the same. However, the problems are eminently different. Due to the stowing

characteristic of the concrete, which has been previously mentioned as the tendency to bridge or "log-jam," the valves and connecting passageways are very large and as free as possible from quick changes of direction or any change of size. The valve passages are nearly the same size, and sometimes even larger than the diameter of the cylinder. When the concrete is particularly stubborn, or for pumping conditions of extreme height or distance, considerable power is required to operate the very large valves. The "stowing" characteristic of the concrete is taken advantage of in the valving operation as the valves are not completely closed in the shut-off position. Stowing of the concrete at the restriction completes the shut-off action.

The piston does not come in direct contact with the concrete nor with the pump cylinder walls. An enlarged replaceable rubber piston-end takes the brunt of pushing the concrete. The piston skirt and cylinder wall back of the rubber piston end are continuously rinsed with washwater. The rubber piston-end keeps the water out of the concrete.

All parts of the pump coming in direct contact with the concrete are subject to considerable wear. As far as possible, liners and other readily replaceable parts are used. The wear is minimized by employing very high grade heat treated materials. Naturally, some concrete mixtures are more abrasive than others, and also, the abrasive action increases with an increase of pumping distance or height.

The pumps most used to date have a cylinder diameter of 190 m.m. (7½ in.) and a stroke of 12 in., and, at normal speed of 48 to 50 r.p.m., have a capacity of 22½ to 30 cu. yd. per hour per cylinder, depending principally upon the workability of the concrete and to some extent upon the combination of height and distance that it must be moved.

A larger standard pump has been developed for use with coarse aggregate of 3 in. maximum size. This has a bore of 200 m.m. (7⅞ in.) and a stroke of 12 in. 8-in. pipe is normally used with this pump. A novel type of inlet valve is used and this valve construction, combined with changes in the structural base, permits lowering the hopper nearly 1½ ft., as compared to the smaller capacity standard pumps.

One of the astonishing things about the Pumpcrete system is the relatively small amount of power needed. Single cylinder pumps are equipped with 30 h.p. variable speed electric motors, or 40 h.p. gas engines—the double pump with a 50 h.p. electric motor or 60 h.p. gas engine. The power cost is rarely more than one cent per cu. yd.

For low slump or harsh mixtures, and wherever segregation is likely to occur in charging the hopper, as for example, when using a belt conveyor or a series of chutes, a pug-mill type of remixing hopper is used instead of a simple cone type hopper. This adjunct has considerably broadened the range of concrete mixtures successfully handled. Even with very pumpable mixtures, it is of considerable benefit, if long delays are encountered, as it keeps the concrete in perfect condition until it is sucked into the cylinder.

The 200 m.m. (7⅞ in.) Pumpcretes are occasionally equipped with a rotary drum remixer which has a loading height of approximately 4 ft. to 5 ft. for a single pump, and 5 ft. to 6 ft. for a double pump, depending on type. The rotary drum remixer has so far been used only for tunnel lining work, but undoubtedly has a broad application for readily portable unit.

PIPE LINE

After the concrete leaves the pump, it moves through the pipe in synchronous impulses, but at all times the pipe is completely filled and there is no tendency to segregate or disarrange the structure of the concrete mixture. There is a tendency due to the troweling action of the inside of the pipe to bring a slight surplus of grout to the outside of the concrete stream which is probably very helpful to pumping as the grout is the only part of the concrete that can possibly act as a lubricant. Due to the remixing action of the pump itself and freedom from segregation in the pipe line, the concrete as it reaches the end of the line is normally in a better state of uniformity of mixture than it was in the hopper of the pump.

Even after the concrete has passed through the pump into the pipe line, it is still a stubborn material to handle. The pipe line is usually very large, frequently almost as large as the pump cylinder diameter, as any change in the contour or size of the passage is likely to cause trouble. Even an enlargement will cause trouble at the point where the normal size is resumed. Large radius curves can be readily negotiated, but add considerably to the pipe friction losses. As an approximation, a 10 ft. piece of pipe bent to form a 5 ft. radius 90 degree elbow has about the same resistance as 40 ft. of straight pipe. In our experimental tests we can develop a very substantial resistance even with a short pipe line by using bends totalling 8 or 10 right angles. In pumping through pipe lines smaller in diameter than the pump cylinder, a taper section is employed to accomplish the reduction. For difficult concrete mixtures, a reduction from 8 in. to 7 in. is made in approximately 6 ft., and from 7 in. to 6 in. in 10 ft.

FIG. 2—TYPICAL PLANT SET-UP SHOWING TWO 28-S MIXERS, DOUBLE "PUMPCRETE" AND PIPE LINE. NOTE THE QUICK ACTING TOGGLE COUPLINGS

Perfecting a practical Siamese "Y" connection to combine the two streams of a double pump was the result of much study and experimentation. The present construction involves an enlargement at the junction of the two incoming pipes so that the area at the point where the streams are completely merged is one and one-half to two times the area of either of the incoming pipes. To reduce this to normal pipe size involves the use of one or more taper sections connected in series.

The pipe line itself is of unusual construction with special toggle couplings that can be connected in half a minute and disconnected in 10 seconds.

The standard length of a section is 10 ft. In addition, 5 ft., 3 ft., and 1 ft. sections are furnished, also 90, 45 and 22½ degree bends. With this assortment, almost any desired point can be reached.

6-in., 7-in. and 8-in. pipes are used. The nominal size is the outside diameter, and the wall thickness is approximately 3/16 in. or less.

The wear in the pipe line is surprisingly small. Most of the pipe, including the bends, will be good for more than 50,000 cu. yds.

It is frequently advisable to have sufficient pipe for a number of "artery" lines in different directions laid down at all times. On some of the Mississippi river lock and dam jobs, approximately half a mile of pipe was available, although the maximum pumping distance rarely if ever exceeded 800 ft.

A general idea of present Pumpcrete performance limitations, given by the following table of Pipe Line Data, is based on reducing the pumping resistance of any combination of straight pipe, bends and differences in elevation to the resistance of an equivalent length of straight horizontal pipe.

PIPE LINE DATA

Pipe Size	Equivalent Length of Straight Horizontal Pipe*	Vertical Distance**	Maximum Aggregate Square Screen Size
WITH PUMPCRETE 200 (8 In.)			
8″ O.D.	1000 ft.	100 ft.	3″
7″ O.D.	800 ft.	100 ft.	2½″
WITH PUMPCRETE 190 (7½ In.)			
7″ O.D.	1000 ft.	100 ft.	2½″
6″ O.D.	600 ft.	100 ft.	1½″

*A 90 degree bend is figured as equivalent to 40 ft. of horizontal piping, a 45 degree bend equivalent to 20 ft., and 22½ degree bend equivalent to 10 ft.
**Not over 200 ft. of pipe, actually.

A combination of horizontal and vertical distances is to be calculated on the basis of 1 ft. of vertical equalling 8 ft. of horizontal pumping. The total equivalent distance should not exceed the equivalent horizontal pumping distances shown in the tabulation.

CLEANING THE PIPE LINE

It was the common practice in Europe until recently to disconnect the pipe sections one at a time, stand the sections up on end to let the concrete slide out and then flush the inside either with a hose or a swab. The pipe line was then reconnected, ready for the next pour.

A more practical, quicker and less wasteful method is used on all of the American Pumpcrete jobs. Cleaning of the pipe line is accomplished by the use of a "go-devil," which is propelled through the line by liquid or air pressure. The go-devil is a dumbell shaped piece with a cup rubber on each end. These rubbers are a close fit in the pipe and with the cups turned toward the liquid, the seal is the same as in a simple plunger pump, like a tire pump.

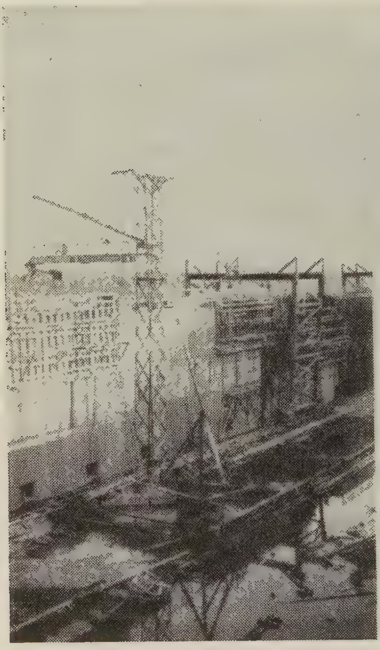

FIG. 3—DISTRIBUTING TOWER DEVELOPED BY CONTRACTOR, MISSISSIPPI RIVER LOCK 22

FIG. 4—DOUBLE PUMPCRETE PUMPING CONCRETE FOR POWER HOUSE AT BOULDER DAM LOADED FROM 8 CU. YD. CABLE-WAY BUCKET

The procedure is as follows: At the completion of the pour, all the concrete in the hopper is pumped out into the line. When pumping against a vertical head, a shut-off valve in the line near the pump is closed and the line adjacent to the pump is opened by removing one or two sections. In the case of the double Pumpcrete, the Siamese and taper sections are removed. A short section of pipe is attached to the near end of the pipe line and a wad of excelsior or sacks is pushed into this pipe, followed by a go-devil; then another wad of excelsior or sacks and another go-devil. The end of the pipe is then capped with a piece containing a hose connection.

In the meantime Pumpcrete hopper and pump are flushed out and any stones or rocks in the pump are scraped out with a suitable tool. After the pump is thoroughly washed out, it is converted into a high pressure water pump by the addition of water valves above the inlet concrete valves and beyond the outlet concrete valves. This converted pump is then connected to the pipe line cap by a high pressure hose. The hopper of the Pumpcrete is next filled with water and pumping

of water is started. The water pressure propels the go-devils, which in turn push the concrete in the pipe, and as the go-devils advance, the concrete is forced out of the end of the line into the forms (or wasted if a bad guess has been made as to the amount required). The location of the go-devils can be determined accurately by tapping the pipe, and pumping is stopped before the go-devil reached the end of the line, so that there is no danger of flooding the pour. Of course the water back of the go-devil does a very fine job of rinsing out the pipe.

This probably sounds like a long and tedious task. Actually the time lost is only a matter of 5, 10 or 15 minutes. The least time is lost with a single pump equipped with a cone hopper. Most time is required for a double pump with a remixing hopper.

An auxiliary pump with a capacity of 80 gals. per minute or more and capable of developing a pressure of 500 p.s.i. can be used for pumping out. This saves considerable time as it is not necessary to wait while the Pumpcrete is being flushed out and being converted into a water pump. With an auxiliary pump only 2 or 3 minutes time is lost.

The pipe line can be successfully cleaned by propelling the go-devil by air pressure. This is the customary procedure in tunnel lining work and is occasionally necessary where it is difficult to dispose of the water that would be used in cleaning the line. For short lines a tight wad of wet sacks can be used in place of the go-devil as the air that gets past into the concrete does no particularly damage. The action in cleaning out is entirely different than when using water, as due to the expansion of the air, the go-devil may get out of control and it is important that the end of the line be pointed at some solid or yielding object that cannot be injured by the impact of the concrete or go-devil.

DISTRIBUTING DEVICES

A considerable number of standard accessories or "gadgets" have been developed in connection with the pipe line to facilitate placing. There are several kinds of spouts and chutes that range all the way from a very small affair that hangs from a bolt supported on the end of the pipe line, to a very elaborate full swivel self-supporting wheel-mounted spout that receives concrete from a pipe line that is supported independent of the forms on an "A" frame or equivalent. A full swivel connection is also available, which permits the end of the pipe line itself to be moved in a full circle. For specialized work such as putting concrete in walls that must be brought up in lifts of 18 in. or 2 ft., or more, to comply with the specifications, bottom drop gates in the pipe line itself are available.

One standard accessory that is interesting on account of its simplicity, is the shut-off valve which consists of a short section so arranged that pins can be driven in through holes in one side. These pins do not completely close the passage, but the same condition of "stowing" is effective there, as in the valves of the pump itself. The shut-off valve is used wherever vertical pumping is involved, either up or down. For pumping up, the valve is usually located close to the pump. For pumping down it is in the lower horizontal line.

For the San Francisco-Oakland Bay Bridge job, it was necessary to use hose sections in the line at various places because the pumping plant was on a barge and the caissons in which concrete was being pumped were anchored in the mud, and the hose sections took care of the relative movements due to tides and waves. These hose sections were of unusual construction and were capable of withstanding pressure of 700 p.s.i., or probably considerably more. Similar hose sections are sometimes used at the end of the pipe line to considerable advantage in placing the concrete just where it is needed.

GENERAL ENGINEERING INFORMATION

It is with considerable hesitation that I approach the next subject. Our present data are too meager, too inadequate to submit as a basis of engineering determination. In spite of that, some of the general relationships are well defined, and the trends disclosed are helpful to an understanding of the general problem of transporting concrete by pipe line.

An almost infinite variety of concrete mixes are possible, and, even with given proportions of all of the solid constituents, the characteristics change a great deal with a difference in water content. Consequently, it is only possible to speak of the action of different mixtures in the pipe line in general terms. We know that drier mixtures require a greater pumping force than wetter ones. On the other hand, some very dry mixtures are easier to pump than others at their best consistency.

In the fall of 1932 we conducted a number of tests on the first double Pumpcrete before it was shipped to Boulder Dam. These tests and some additional data secured at Boulder Dam and on the New York Central Viaduct job in New York served as the ground work for the values presented. Our experiences on a great variety of work since then have not been contradictory.

The volumetric efficiency of the system, i. e., ratio of the volume of concrete pumped to the piston displacement, is affected principally by (1) consistency and workability of the mix; (2) pumping height and distance.

If concrete is readily pumpable, and the slump is favorable, (2 in. to 6 in.), the volumetric efficiency rarely falls below 80 per cent. For medium and short distance horizontal pumping, the efficiency will frequently exceed 90 per cent, sometimes 95 per cent. If concrete is too harsh for pumping, or too dry, the average efficiency may be in the seventies, and, in extreme cases, the efficiency may drop to as low as 30 per cent, and yet pumping can be continued with reasonable certainty.

The pump speed has very little effect for the customary range between 40 and 50 r.p.m. For slower speeds, if pumping against a considerable head, the effect is adverse, due to back slip through the valves while they are being opened or closed. An excess speed can interfere with the complete filling of the cylinder on the suction stroke, particularly with dry concrete. Inadequate head of concrete above the inlet valve may have the same effect as excess speed.

The values given in the table of Pipe Line Data were first predicated on the following approximate relationship of pipe line pressures and slump as determined for single and double pumps of 180 m.m., ($7\frac{3}{32}$ in.) bore and 12 in. stroke.

PRESSURE DROP POUND PER SQUARE INCH FOR 100 FEET OF STRAIGHT LINE
(Approximate Average)

Diameter of Pipe	5 In.	6 In.
6 in. Slump	40	25
2 in. Slump	70	40

Pressure due to lift or static head is one pound per square inch for each foot vertical and is substantially independent of the slump.

Due to the practical difficulties of securing accurate pressure readings, we know comparatively little about the pressures in the pump but have reason to think that these pressures may be as much as 500 p.s.i., although, for average pumping conditions it is probable that the pressures are not much more than half that. The water pressures required to move the go-devil can be measured. Usually the pipe lines can be cleaned when using the go-devils, with pressures of much less than 500 p.s.i.

As the concrete flow in the pipe line comes to complete rest after each pressure stroke with a single pump and nearly so with a double pump, much energy is required to accelerate the mass for each pressure application, and that energy is proportional to the mass, and independent of the slump.

Obviously the power required is dependent on both pipe friction and inertia, and while there is a substantial increase in power requirements with the drier mixes, the increase is by no means proportional to the increase of pipe friction. To illustrate the general relation, the best record with 1½ in. slump, 4½ bag concrete, is about 1100 equivalent ft. while the best record with 6 in. or 7 in. slump, 6-bag concrete is more than 1400 ft. of actual pipe. Both of these records are far beyond the guaranteed performance as shown in the table of Pipe Line Data.

There is no mathematical justification for the statement accompanying that table that 1 ft. of vertical equals 8 ft. of horizontal pumping. It is an approximation that satisfies the lay mind and saves a lot of argument. In a practical way it works and it is close enough to the facts to permit successful planning for limit applications. From mathematical considerations, it is evident that if the pipe friction with 2 in. slump is 75 per cent more than with 6 in. slump, and the pressure due to the head is independent of the slump, a correct conversion factor must be a variable and not a constant. Other factors such as pipe line velocity have a bearing and a complete formula taking all of the factors into consideration would probably be too complicated to be of practical value.

As yet we have not determined that the proximity to, or remoteness from the pump of the vertical pipe or lift has any important bearing on the limits of operation or capacity. Due to the synchronous starting and stopping of the flow, it is advantageous to have a horizontal run of pipe, say 30 to 50 ft., between the pump and the lift, as the inertia effects of the concrete stream in the horizontal pipe to some extent counteract the back pressure due to the vertical head, permitting the outlet valves to close with less effort required and less wear. In explanation of the foregoing, the concrete stream in the pipe acquires a substantial velocity at the middle of each pressure stroke, and this velocity has a tendency to continue even after the piston has reached the end of the stroke. The inertia of the decelerating mass tends to offset or minimize the back pressure due to the vertical head.

CONCRETE MIXTURE

Nearly all of the concrete mixtures commonly used are pumpable, provided they are of a readily workable consistency. Workability in a practical way is gauged by the ease of flowing, puddling or vibrating the concrete into final place in the forms. Most of the factors that make for workability also promote pumpability. However, a concrete may be workable in the sense that it will readily tumble or landslide

into place, but it does not always follow that the same mixtures will respond satisfactorily to pressure. No amount of pressure will pump stone, gravel or sand, either individually or collectively, unless the solid pieces are "floated" in a great surplus of fluid, or, as in the case of concrete, the solid ingredients are vitalized into a fluid mass by the lubricating properties of the cement paste.

With this conception, it seems to be important to consider concrete mixtures, not only from the standpoint of workability, but more particularly as to the kind of workability, namely, "gravity workability" and "pressure workability", as characterized by the types of activating force to which they respond. Except for the drier mixtures, the distinction is of minor consequence, but for the large amount of specification concrete with low water content, which must be vibrated in place, there is a decided difference.

There may be other answers, but from our observations it seems that pressure workability requires more sand than gravity workability. Just adding water will not make up for a deficiency in sand, although adding water and cement together, always helps. Apparently, enough sand and cement paste is required to minimize particle interference of the coarse aggregate pieces. Also, it seems that a deficiency of fines in the sand has a greater effect on pressure workability than on gravity workability. Fortunately, nearly all of the later specifications are giving proper consideration to the fines. This subject of workability has been most ably developed and discussed by T. C. Powers in his paper, "Studies of Workability of Concrete."* These studies were made before the general use of internal vibrators, and it is probable that the values for optimum per cent of sand are more favorable to pumping of concrete than some of the present practices. The remolding test used as the measure of plasticity comes closer to giving an indication of pressure workability or pumpability than any of the other standard tests. Nevertheless, it is principally a gravity workability test. The conclusions, however, are definitely in agreement with actual experiences and observations of commercial concrete pump applications, and should be recognized in designing concrete mixtures for Pumpcrete placing.

Table 1 gives detailed information covering the concrete mixtures used for a considerable number of jobs. In some cases the information is quite complete, in others, only the weights of cement, sand and coarse aggregate are now available. It is to be understood that the exact proportions were subject to frequent adjustment, and usually, continued experimentation to compensate for the variations of the

*Journal, Amer. Concrete Inst., Feb. 1932, *Proceedings*, Vol. 28, p. 419.

TABLE 1—DETAILS OF CONCRETE MIXTURES PLACED BY PUMPCRETE ON REPRESENTATIVE JOBS

Job	1 Boulder Dam	2 Huey Long Bridge	3 Missis-sippi River Lock No. 4	4 Mil-waukee Via-duct	5 Green River Dam	6 San Francisco Oak.-Bay Bridge	7 Missis-sippi River Dam No. 5	8 Missis-sippi River Dam No. 5	9 Missis-sippi River Lock No. 22
Year	1932	1933	1933	1933	1933	1934	1934	1934	1934
Yardage	See 11	17,000	55,000	5,200	55,000	75,000	40,000	See 7	80,000
Cement	500	470	470	564	470	470	435	437	423
Admix.									25
Sand	1000	1320	1270	1235	1130	1331	1250	1199	1270
Coarse Aggregate									
−½	x	x	x	x	x	x	x	x	x
−¾	757	x	x	720	x	512	x	1091	x
−1	x	x	x	x	x	x	1140	x	920
−1¼	x	x	x	x	x	x	x	x	x
−1½	1400	1950	x	970	2030	615	x	1250	x
−2			x	x		x	1140		1270
−2½			2170	345		956			
Water					273	266	232		244
Sand	Good	Bad	Good	Good	Fair	Fair	Blend	Blend	Bad
Coarse Aggregate	Gravel	Gravel	Gravel	Gravel	Gravel and Cr. Stone	Gravel and Cr. Stone	Gravel	Gravel	Cr. Stone
Slump	2 − 5	1 − 4	2 − 5	6	1½ − 2½	1½ − 3	1	1½	1
Pumpability	Fair	Fair	Good	Good	Good	Good	Good	Bad	Bad

SIEVE ANALYSIS OF SAND—PER CENT PASSING

	1	2	3	4	5	6	7	8	9
⅜								100	
4		99.5			99.1			96.2	97.2
8		95.5			87.8			81.4	88.5
16		84.0			75.4			65.4	70.6
30		66.0			55.2			36.0	27.1
50		5.0			14.0			11.3	6.9
100		1.0			1.5			2.0	.5

"x" Indicates that some of this size is included with the weight given below it.

aggregates. Each of the mixtures shown met satisfactorily the requirements of the particular job for workability and strength.

The following comments throw some light on the reasons for the classification as to pumpability, the numbers corresponding to the columns of Table 1.

(1) This was a border line mix sometimes O. K., sometimes very difficult when the pea gravel varied. See Column 10 for a better proportion of the same materials. Probably with the remixing hopper later developed, this mix would have been entirely satisfactory.

(2) This was a "temperamental" material to pump, particularly in hot weather, probably due to the poor quality of the sand. The concrete bled badly and best results were obtained at about 3 in. slump.

(3) Plenty of good sand—O. K.

(4) Plenty of cement, sand and water. Good gravel. High strength and very easy to pump.

(5) With 5 bags of cement, this material was entirely satisfactory.

(6) This concrete was delivered to the Pumpcrete hoppers by belt conveyors.

TABLE 1 (PART 2)—DETAILS OF CONCRETE MIXTURES PLACED BY PUMPCRETE ON RESPRESENTATIVE JOBS

Job	10 Boulder Dam	11 Boulder Dam	12 Missis-sippi River Lock No. 5A	13 Missis-sippi River Lock No. 8	14 Missis-sippi River Lock No. 16	15 Metro-poli-tan Tunnel	16 Joe Wheeler Dam	17 Joe Wheeler Dam	18 Walsh
Year	1934	1934	1934	1934	1934	1935-6-7	1935	1935	1935-6
Yardage	See 11	250,000	60,000	65,000	70,000	600,000	25,000	See 16	150,000
Cement	480	445	423	432	423	517	455	480	564
Admix.									
Sand	1180	1032	1180	1185	1399	1110	1135	1250	1080
Coarse Aggregate									
−½	x	x	x	x	x	100	x	x	126
−¾	851	820	x	x	x	x	630	640	x
−1	x	x	1392	1155	849	x	x	x	x
−1¼	x	x	x	x	x	860	x	x	1010
−1½	1281	700	x	x	x	x	835	1280	x
−2		x	928	1155	1273	x	x		x
−2½		x				1180	x		854
−3		840					625		
Water		259	245	247		316			346
Sand	Good	Good	Good	Good	Blend Good	Fair	Bad	Bad	Blend Fair
Coarse Aggregate	Gravel	Gravel	Gravel	Gravel	Cr. Stone	70% Gravel	Gravel and Cr. Stone	Gravel	Gravel and Cr. Stone
Slump	4 − 6	4	1½	1½	1 − 1¼	4	1½ − 4	2½ − 4	4
Pumpability	Good	Good	Good	Good	Bad	Fair	Bad	Fair	Good
SIEVE ANALYSIS OF SAND—PER CENT PASSING									
⅜			100	100					
4			98.9	96.2		100		99.5	
8			88.0	88.8		84		84.5	
16			65.5	68.0		62		68.5	
30			26.8	31.9		39		46.5	
50			11.7	9.4		22		5.5	
100			5.6	1.3		10		.5	
200						3			

"x" Indicates that some of this size is included with the weight given below it.

The remixing hoppers on the Pumpcrete would correct most but not all of the segregation from the belts but would not make up the loss of grout when the belt scrapers failed to function. Until the mix was proportioned as shown, trouble was encountered frequently. After that, the performance was very dependable and satisfactory.

(7) This concrete was splendid and results were equally so.

(8) This is the same job some months later. 2-in. aggregate no longer available. With 1½-in. maximum aggregate, it was very difficult to develop the specified strengths, using 4½ bags of cement, without being too dry to pump.

(9) Crushed stone concrete needs more sand, cement and water than gravel concrete for equal workability, but the specifications were the controlling factor.

(10) Wonderful concrete to pump or chute.

(11) This concrete is much harder to pump than 10, but good, dependable results were obtained. More sand would have made the job easier.

(12) Enough of the larger coarse aggregate reduced the sand requirement so the cement and water were adequate.

(13) The contractor was smart enough to use 9 lb. extra cement and secured fine workability and high production.

(14) Another crushed stone job. The sand was fine which helped. Would have been easy to pump at 3-in. slump, but with slumps of less than 1½-in. was "plenty tough." With the same water and cement limitations, crushed stone is more difficult to pump than gravel concrete. However, with enough extra sand to compensate for the difference in voids, and extra cement and water to correspond, crushed stone concrete is just as pumpable as gravel concrete.

(15) Desert materials are converted into adequate concrete. Not hard to pump but compare the water content to any of the lock and dam jobs. Of course, the materials are usually bone dry.

(16) and (17) Not enough fines in the sand.

(18) Similar to 15.

This matter of concrete mixtures is much too complex to permit generalizing, but it does seem that there is one conclusion that can be drawn that applies generally, and that is: provided there is enough good sand and the slump can be maintained at 2 in. or more, the results will be consistently satisfactory. In some cases "enough" sand may be more than the concrete technician approves of and with less than 4¾ bags of cement per yard some clever juggling is sometimes required to combine adequate strength and workability. With 3 in. stone and minimum sand the slump should be 3 in. or more.

SUMMARY

Concrete is actually pumped by a piston pump in much the same manner as water. The Pumpcrete machine is especially designed for this purpose, with a supply hopper surmounting a horizontal cylinder and very large inlet and outlet valves mechanically operated, in timed relation with the movements of the piston and crank shaft. Due to the unusual characteristics of the material being handled, it is not necessary to close the valves completely so there is no "nut cracker" action. Instead they are closed only partially and the concrete "stows" at the restriction, completing the closure. No air is used in connection with the pumping action and the pump cylinder and pipe line are completely filled with concrete.

Concrete is delivered from the hopper to the forms in a uniform condition of mixture and free from segregation.

The pump parts that come in direct contact with the concrete and are subject to wear are readily replaceable, but due to proper choice of materials and proper processes of manufacture, have a surprisingly long life.

Pumpcrete machines are now built in single and double cylinder models, with capacities ranging from 20 or 25 to 65 cu. yd. per hour. A 50 h.p. electric motor, or 60 h.p. gasoline engine is used on the largest machine, and power cost rarely exceeds 1¢ per cu. yd.

The pipe line is usually nearly as large in diameter as the pump cylinder. It is made in standard interchangeable sections, with quick-acting toggle couplings and rubber gaskets to prevent leakage. Due to the low velocity of flow, pipe wear is negligible.

The pipe line is cleaned at the end of a pour and all the concrete in the pipe line is expelled into the forms by inserting a go-devil in the line, just in front of the pump and forcing the go-devil and concrete through the line by water or air pressure.

The Pumpcrete system has been used for a great variety of jobs, with yardages as little as 1000 and as much as 80,000, not counting Boulder Dam where approximately one quarter million yards have been pumped.

The cement content varied from 4 bags per yard to 6. Much of the concrete pumped was approximately 4½ bags per yard and dry enough to develop strength in excess of 3200 p.s.i.

The minimum sand content was 1000 lb. per yard, and the maximum was 1400 lb. and more. The coarse aggregate varied from pea gravel up to and including anything that would pass a 3 in. square screen.

The slump varied from ½ in. up. Probably the most dependable slump is about 3 in., but the maximum distance and height is obtained with good concrete of 6 or 7 in. slump.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Slabs Supported on Four Sides*

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The proposed new "Building Regulations for Reinforced Concrete" (see this Journal, *Nov.-Dec. 1935, p. 181 and supplementary publication there announced) as recommended for adoption by Institute Committee 501, Standard Building Code, include a complete revision of existing Code requirements for the design of two-way slabs which are supported on four sides. To facilitate discussion when the proposed new code should be formally presented for action of the Convention (Chicago, Feb. 25-27), Mr. DiStasio, a member of Committee 501, was invited to write a brief paper outlining the regulations and basic formulas as prescribed by the Committee. The present paper, in which Mr. van Buren has collaborated, and written with the advice and suggestion of other members of Committee 501, as acknowledged by the authors, is the result.*—Editor

THE PROBLEM OF TWO WAY SLABS

The mathematical analysis of slabs supported on four sides is a problem of considerable intricacy. Bending moments are affected not only by the complex distribution of the stresses within the panel itself, but also by variations in the length, stiffness, and loading of adjacent bays. Notable articles ([1], [2]) in recent years have established a basis for a better understanding of the actions controlling the behavior of this type of structure. Technical methods are available for analyzing particular situations by close approximation, and formulas have been proposed for dealing with a series of equal adjacent panels in various arrangements of continuity. However, a building with all panels of the same size and shape is rarely encountered in practice, and a means of handling the usual irregularities is sorely needed by the profession. To be practical, a code should provide for all contributory factors in the simplest manner possible. For two way slabs, this is best ac-

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([1], [2]) See references at end of paper.

complished by the use of one general method for all conditions of rectangularity, restraint, arrangement of loading, and variations in adjoining bays.

THE EQUIVALENT UNIFORM LOAD METHOD

It is the intent of the two way slab regulations of the proposed Standard Building Code to provide one universal formula for bending moment to cover all conditions and situations affecting the behavior of a slab in the usual building structure. For a unit width of slab, this formula may be written

$$M = \frac{1}{f} e_A r_A w A^2 \qquad (1)$$

in which $\frac{1}{f}$ is the bending moment coefficient,

A is the span in direction considered,

w is intensity of uniformly distributed loading on the panel,

r_A is the proportion of total load carried by span A.

Except for limiting cases, the total load on span A, $r_A w A$, is not uniformly distributed along the span, but, when modified by the factor e_A, a total equivalent uniform load, $e_A r_A w A$, is obtained from which the bending moment is found directly by formula (1).

A formula of the same general type obtained by replacing $e_A r_A$ with $1 - e_A r_A$ governs the design of the parallel supporting beams. End shear follows directly from the total load carried by the span, being, when end moments are equal, $\frac{1}{2} r_A w A$ for slabs and $\frac{1}{2}(1 - r_A) w A$ per unit of tributary width for the parallel beams. Approximate formulas for the intensity of loading are given from which shears at intermediate points may be calculated. A minimum thickness formula, based on limiting deflection to definite ratios of the span is established by which all cases of rectangularity and continuity are made consistent. Finally, rules are formulated to control the arrangement of the reinforcing.

The regulations are based on the following principles:

(1) The greater proportion of the panel load is carried in the stiffer of the two directions of the panel.

(2) The proportion of the total load carried by any element is generally not uniformly distributed along its span.

(3) The slab and parallel beams together carry the total load.

DISTRIBUTION OF THE TOTAL PANEL LOAD

The distances between the lines of inflection, when uniform load is applied to the span under consideration only, form convenient measures of the relative stiffness of the two directions. In a panel, $A \times B$, these

distances are given by $F_A A$ and $F_B B$ respectively. It is found that F_A and F_B, thus defined, vary very little with changes in spans, and that formulas therefor, derived on the basis of an infinite series of spans all of which are alike except the loaded one, are sufficiently accurate for other arrangements of the more distant spans. Derivations, given in the Appendix I, result in the following equations in which K_A is the stiffness factor $\frac{I}{A}$ for the span considered, and K_{AR} is the stiffness factor of the adjacent span.

Simple Span, $F_A = 1$ .... (2A)

End Span continuous one end only, $F_A = 1 - \dfrac{.25}{1 + \dfrac{7K_A}{8K_{AR}}}$ .... (2B)

Interior Continuous span, $F_A = \sqrt{1 - \dfrac{1}{1.5 + \dfrac{7K_A}{8K_{AR}}}}$ .... (2C)

For interior spans, when adjacent bays to right and left are different, F_A is to be taken as the average value. Similar equations determine the values of F_B by substituting the stiffness factors of the B spans. Results of the equations are tabulated in Table 1.

TABLE 1

$\frac{K_A}{K_{AR}}$	0	.25	.50	.67	.80	1.00	1.25	1.50	2.00	4.00	Infinity
F_A, End Span	.75	.80	.83	.84	.85	.87	.88	.89	.91	.95	1.00
F_A, Interior Span	.58	.65	.69	.72	.74	.76	.78	.80	.83	.89	1.00

An inspection of the table demonstrates the relatively narrow range of variation of these factors. Therefore, for all practical purposes, when the ratio of the stiffness factor of the span considered to each adjacent span is at least 2/3 or at most 3/2, F_A or F_B may be taken without recourse to the formulas as .76 for interior spans, .87 for end spans, and 1.00 for simple spans.

The proportion of total panel load carried in one direction, r_A, is then found from the third power formula

$$r_A = \frac{1}{1 + \left(\dfrac{F_A A}{F_B B}\right)^3} \qquad \text{(3A)}$$

and in the direction at right angles,

$$r_B = \frac{1}{1 + \left(\dfrac{F_B B}{F_A A}\right)^3} = 1 - r_A \qquad \text{(3B)}$$

Of the various generally recognized distribution formulas, e.g., straight line, fourth power, third power, etc., the third power formula was chosen after an exhaustive study as giving results most consistent with the investigation.

THE EQUIVALENT UNIFORM LOAD

In addition to distributing the total load between the two directions of the panel, proper consideration must be given to the variation in the intensity of loading along each span. Tests and derivations (2) (3) agree that in a simple square panel, the maximum slab moment is $\frac{1}{24} wA^2$. As one half the total panel load is carried in each direction, this indicates that the intensity of loading varies in such a manner that the moment produced is 2/3 of the moment which would obtain were the same load uniformly distributed. In a simple rectangular panel in which the long side is twice (4) the short side, it is evident that nearly all the load is carried by the short direction, and the panel approaches the condition of one way construction with uniformly distributed load. Between these limits, therefore, the ratio of equivalent uniform load to total load, e_A, must vary between 2/3 and 1. Similarly, in continuous panels, the ratio will vary through the same range when $F_A A$ and $F_B B$ are substituted for the complete spans. The exact curve of variation between 2/3 and 1 is relatively unimportant, but the formulas

$$e_A = \frac{2}{4 - \dfrac{F_B B}{F_A A}} \qquad \text{(4A)} \quad \text{and} \quad e_B = \frac{2}{4 - \dfrac{F_A A}{F_B B}} \qquad \text{(4B)}$$

gave results most consistent with the studies made. The equivalent uniform loads to be used in determining slab moments are therefore

$e_A r_A w$ and $e_B r_B w$, and with these values, calculations may proceed as in one way construction.

The formulas given for e_A and e_B contemplate that the edges of the panel are monolithic with or securely anchored down to the supports. When freely supported, as explained later in the discussion of corner moments, the corners of the panel tend to lift up and stress is concentrated towards the center of the panel. Quite the reverse of the more usual anchored condition, this concentration of stress becomes worse as more load is applied. To provide for this, the regulations specify that when either one or both ends of a span are not rigidly attached to the supports, the value of e in that direction shall be taken as 1.

BENDING MOMENT COEFFICIENTS

Bending moment coefficients, $\frac{1}{f}$, may be found by any of the recognized methods for analysing a continuous structure by loading the successive spans in either direction with their equivalent uniform live and dead loads. The position of the live loads should be so varied as to produce the maximum moments at the controlling sections. If, however, arbitrary maximum bending moment coefficients are permitted for slabs under other sections of the Code, these may be used subject to the usual limitations governing such coefficients.

SLAB SHEAR AND INTENSITY OF LOADING

In the design of solid slabs, determination of the maximum end shears are usually sufficient. As previously explained, these are direct functions of the load distribution, being $\frac{1}{2} r_A w A$ and $\frac{1}{2} r_B w B$ when end moments are equal. However, it is sometimes necessary, particularly in ribbed construction, to evaluate the maximum shears at intermediate points. For this purpose, a knowledge of the variation in intensity of loading along each span is essential. Exact calculations would be considerably involved, and, in the interest of simplicity, the approximation of substituting two straight lines for the true curvilinear variation affords an easy method of obtaining the results desired with satisfactory accuracy. With this approximation, formulas for the intensity of loading are derived on the basis that, first, the total moment on the span due to the variable loading is the same as that resulting from the equivalent uniform loading, and, second, the end shear obtained from the variable loading equals $\frac{1}{2} r_A w_A$. The values for the intensity of loading at center and support are given in Fig. 1, and their derivation may be checked from equations 5 and 6.

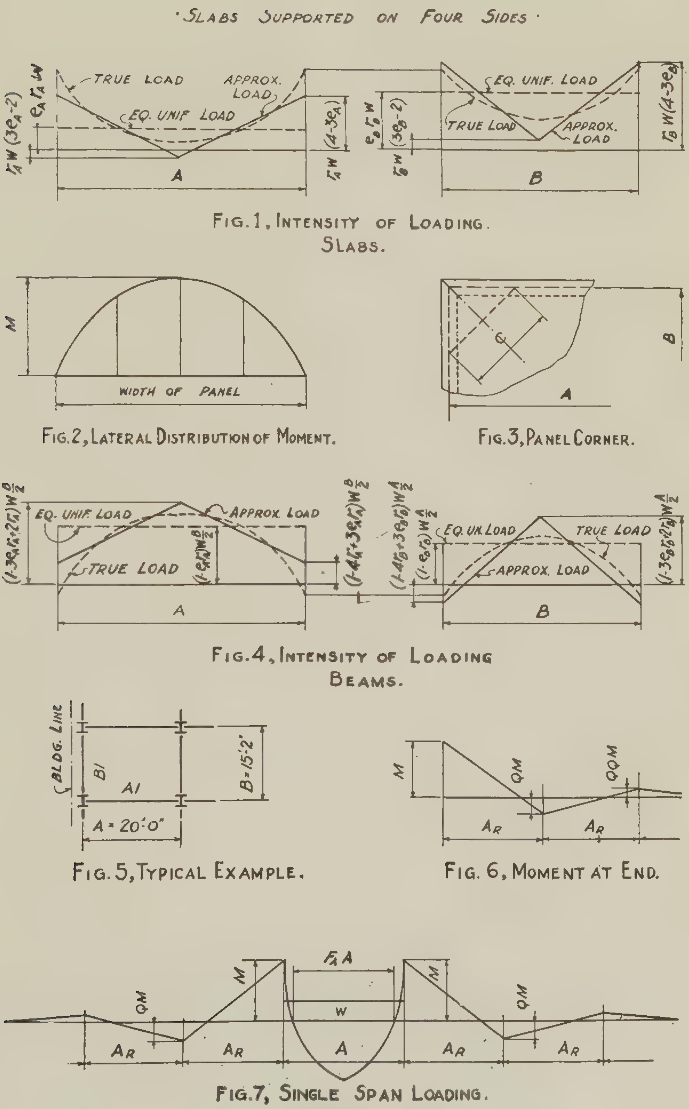
·SLABS SUPPORTED ON FOUR SIDES·
TRUE LOAD
APPROX. LOAD
EQ. UNIF. LOAD
A
B
TRUE LOAD
APPROX. LOAD
FIG. 1, INTENSITY OF LOADING. SLABS.
M
WIDTH OF PANEL
FIG. 2, LATERAL DISTRIBUTION OF MOMENT.
C
A
B
FIG. 3, PANEL CORNER.
EQ. UNIF. LOAD
APPROX. LOAD
TRUE LOAD
EQ. UN. LOAD
TRUE LOAD
APPROX. LOAD
A
B
FIG. 4, INTENSITY OF LOADING BEAMS.
BLDG. LINE
B1
A1
B = 15'-2"
A = 20'-0"
FIG. 5, TYPICAL EXAMPLE.
M
QM
QQM
A_R
A_R
FIG. 6, MOMENT AT END.
$F_A A$
M
M
W
QM
QM
A_R
A_R
A
A_R
A_R
FIG. 7, SINGLE SPAN LOADING.

FIG. 1, 2, 3, 4, 5, 6, 7

$$r_A w\,(3e_A—2)\,\frac{A}{2} \times \frac{A}{4} + r_A w\,(4—3e_A—3e_A + 2)\,\frac{A}{2} \times \tfrac{1}{2} \times \frac{A}{6} = e_A r_A w \frac{A^2}{8} \;\ldots\ldots(5)$$

$$\tfrac{1}{2}\left(r_A w\,(3e_A—2) + r_A w\,(4—3e_A)\right)\frac{A}{2} = \tfrac{1}{2} r_A w A \ldots\ldots(6)$$

While in certain instances, due to the approximations, negative values for the intensity of loading may result near the center of the long span, the shear which is a summation of the loading will be correct at the support and a close approximation in regions away from the support.

LATERAL DISTRIBUTION OF THE BENDING MOMENT

The lateral distribution of the bending moment carried by parallel strips of slab is shown in Fig. 2 (2). For at least the center half width, the bending moment may be considered fairly uniform. With anchored edges, this uniformity increases as load is applied, more strips not previously stressed to capacity gradually coming into action and thus materially increasing the factor of safety. However, in conformity with the fact that the intensity of loading carried by each strip increases as its support is approached, strips parallel to and near the sides of the panel carry correspondingly less load—the sum of the load intensity at any point carried by the A and B spans being always equal to w. The bending moment therefore drops off rapidly near the sides of the panel. This condition is further accentuated by T-flange action of the supporting beams causing compressive stresses in the slab which counteract tensile stresses in the reinforcement. The regulations therefore permit that positive steel adjacent to a continuous edge only and for a width not exceeding one fourth of the shorter dimension of the panel may be reduced 25 per cent. Negative steel, to provide for corner moments, discussed subsequently, must be carried at full value for the entire width of the supports.

CORNER MOMENTS

Fig. 3 represents the condition at the corner of a two way panel. It is apparent that span C across the corner being shorter and stiffer than spans A or B will carry most of the load. In this region, therefore, the principal moments will be positive across the diagonal causing tension in the bottom of the slab in the direction of span C, and negative in the direction of the diagonal producing tension in the top of the slab across span C. Analysis could also be made considering only the A and B directions, but the bending moments found would not be the principal moments as large torsional components would also be involved. With unanchored edges, the rotation about span C as an axis due to bending in the direction of the diagonal tends to lift the corner off the

support causing span C to move towards the center as more load is applied. Edges monolithic with or rigidly attached to supports are therefore conducive to a more balanced stress distribution.

Reinforcement of the corner may be attained by the use of either a diagonal or rectangular arrangement of the steel. But, unless additional diagonal reinforcement is supplied to satisfy the principal moments, the concentration of stress near the slab corners necessitates provision for the full value of the negative moment across the entire length of the support, and precludes cutting bars short or reducing the area of the positive steel in the outer quarters more than 25 per cent.

PLATE ACTION

As in all flat plate structures, the torsional resistance of the slab is a primary factor in securing a favorable distribution of the bending moments. To insure the development of this torsional resistance, and to provide some measure of uniformity between the moments of inertia in the two directions, it is specified that the reinforcement per unit width in the long direction shall be at least one third of that supplied in the short direction. This regulation is of particular importance for thin oblong panels with crossing reinforcement where a slight field displacement of the bars may materially influence the effective depth. Plate action, properly considered, requires adequate resistance in both directions.

MINIMUM THICKNESS

To guard against the obvious dangers inherent in too thin slabs, a minimum thickness formula is prescribed,

$$t_3 = \frac{A + B - 0.1N}{72} \sqrt[3]{\frac{2000}{f_c}} \text{ but at least 4 inches} \quad (7)$$

in which N equals the sum of the lengths of those edges of the panel $A \times B$ supporting continuous adjacent spans. This formula is based on limiting deflection to definite ratios of the span consistent with all conditions of rectangularity and continuity. $\frac{1}{24}$ of the span has long been recognized as a minimum desireable thickness for one way simple spans in wood, steel, or concrete. This, for steel at 16000 p. s. i., agrees with the limit for plastered ceilings; for steel at 18000 p. s. i., the ratio becomes $\frac{1}{21.6}$, but in reinforced concrete, with the assistance given

by the concrete to the steel in regions of low stress, the retention of $\frac{1}{24}$ is justified. In a simple square panel, the equivalent uniform load ratio is 2/3, and for deflection equal to a one way slab of the same span, the required thickness is $\frac{1}{36}$ of the span. Similarly, for equal deflection, a continuous uniformly loaded one way span with $\frac{wA^2}{12}$ at the center requires 85 per cent of the thickness of a simple span, and in two way slabs, this percentage may be decreased to 80. The cube root factor was added by the committee to provide for cases where other than 2000 lb. concrete is used. The consistency of the formula is readily demonstrated, as for example, in the case of simple spans, it varies from $\frac{A}{36}$ for a square panel to $\frac{A}{24}$ when B equals $2A$,—a condition approximating one way construction. Furthermore, a comparison with minimum thicknesses satisfactorily followed in commercial practice shows close agreement. It is of course necessary to also check the thickness as required by the stresses.

THE SUPPORTING BEAMS

The analysis of the supporting beams follows directly from the principle that the slab and parallel beams together carry the total load. For the beam of span A, the equivalent uniform loads $(1—r_A)\,w$ and $(1—e_Ar_A)w$, when multiplied by the tributory width, give the loading per unit length of span to be used in determining end shear and moment respectively. Similar equivalent loads $(1—r_B)w$ and $(1—e_Br_B)w$ are applicable to the beam of span B. In the derivation of these expressions, it is to be noted that no compensation has been made in the design of the beams for the reduction in load carried by the outer quarters of the parallel slab which may amount to 8 1/3 per cent of the total panel load. This omission is justified by the many factors tending to increase the factor of safety in this type of construction, such as dome action from surrounding partially loaded bays, redistribution of stress, and the fact that the controlling sections of both slabs and beams are required to be proportioned for maximum moment conditions which do not occur simultaneously.

For purposes of calculating shears at intermediate points, approximate formulas for intensity of loading are derived on a basis similar to

that for slabs. The values for the intensity of loading at center and support due to one half panel width are given in Fig. 4, and their derivation may be checked from equations (8) and (9).

$$\frac{B}{2}\ (1-4r_A + 3e_Ar_A)\ w\frac{A}{2} \times \frac{A}{4} + \frac{B}{2}\ (1-3e_Ar_A + 2r_A-1 + 4r_A-3e_Ar_A)$$

$$\frac{w}{2} \times \frac{A}{2} \times \frac{A}{3} = (1-e_Ar_A)\ w\frac{B}{2}\ \frac{A^2}{8} \quad \ldots\ldots (8)$$

$$\frac{B}{2} \times \tfrac{1}{2} \left((1-4r_A + 3e_Ar_A)\ w + (1-3e_Ar_A + 2r_A)\ w \right) \frac{A}{2}$$

$$= \tfrac{1}{2}\ (1-r_A)\ wA\frac{B}{2} \quad \ldots\ldots (9)$$

Shear calculated from these loadings will be correct at support, and slightly on the safe side at intermediate points.

DESIGN

The principal factors required in the design of two way slabs and their supporting beams are given in Table 2.

TABLE 2

$\frac{F_AA}{F_BB}$	$\frac{F_BB}{F_AA}$	r_A or $1-r_B$	r_B or $1-r_A$	e_A	e_Ar_A	$1-e_Ar_A$	e_B	e_Br_B	$1-e_Br_B$
1.00	1.00	.50	.50	.67	.33	.67	.67	.33	.67
1.1	.91	.43	.57	.65	.28	.72	.69	.39	.61
1.2	.83	.37	.63	.63	.23	.77	.71	.45	.55
1.3	.77	.31	.69	.62	.19	.81	.74	.51	.49
1.4	.71	.27	.73	.61	.16	.84	.77	.56	.44
1.5	.67	.23	.77	.60	.14	.86	.80	.62	.38
1.6	.63	.20	.80	.59	.12	.88	.83	.66	.34
1.7	.59	.17	.83	.59	.10	.90	.87	.72	.28
1.8	.55	.15	.85	.58	.09	.91	.91	.77	.23
1.9	.53	.13	.87	.58	.08	.92	.95	.83	.17
2.0	.50	.11	.89	.57	.06	.94	1.00	.89	.11

The simplicity of the calculations is best illustrated by a typical example. Given the side wall panel shown in Fig. 5, to determine the slab moments, and the end shears and moments in the supporting steel beams. The unit load is taken at 100 lbs. per sq. ft., and beam B1 in addition carries a wall load of 800 lbs. per lin. ft. Arbitrary moment coefficients for the slab are assumed as $\frac{1}{10}$ for end spans and $\frac{1}{12}$ for

interior spans.

$\frac{F_A A}{F_B B}$ equals $\frac{.87 \times 20}{.76 \times 15.2}$ equals 1.5, and with this as an argument, all factors are obtained from Table 2 as follows:

$$\text{Slab of Span } A,\ M = \frac{1}{10} e_A r_A w A^2 = \frac{1}{10} \times .14 \times 100 \times 20^2 = 560'\ \#/\text{ft.}$$

$$\text{Slab of Span } B,\ M = \frac{1}{12} e_B r_B w B^2 = \frac{1}{12} \times .62 \times 100 \times 15.2^2 = 1189'\ \#/\text{ft.}$$

$$\text{Beam } A1,\ \text{reaction } V = (1-r_A)\ wB\frac{A}{2} = .77 \times 100 \times 15.2 \times \frac{20}{2} = 11681 \text{ lb.}$$

$$M = (1-e_A r_A)\ wB\frac{A^2}{8} = .86 \times 100 \times 15.2 \times \frac{20^2}{8} = 65231 \text{ ft. lb.}$$

$$\text{Beam B1}, V = \left((1-r_B)\ w\frac{A}{2} + 800\right)\frac{B}{2} = (.23 \times 100 \times \frac{20}{2} + 800)\ \frac{15.2}{2} =$$

7712 lb.

$$M = \left((1-e_B r_B)\ w\frac{A}{2} + 800\right)\frac{B^2}{8} = (.38 \times 100 \times \frac{20}{2} + 800)\ \frac{15.2^2}{8} = 33944 \text{ ft. lb.}$$

Experience in design of buildings by this method has shown conclusively that calculations are rapidly made and are susceptible to simple tabulation.

COMPARISONS WITH OTHER METHODS

In the development of these formulas, extensive comparisons have been made with the slab moment coefficients of various codes including the usual third power, fourth power, and straight line distributions, Professor Wise' tables, (2) and Doctor Westergaard's formulas.(1) Positive and negative bending moments for the nine general cases of simple spans, end spans, and interior spans in a series of equal rectangular panels of various ratios were included in the study. Fig. 8 shows the comparison with Doctor Westergaard's work. The close agreement is evident, and the differences may be mainly attributed to different assumptions of the proportion of live to dead load used in selecting bending moment coefficients, and to Doctor Westergaard's allowance for redistribution of stress. Curves, also shown, comparing the equivalent load formulas for Case I with the Elastic Web Method of Professor Wise, show practical coincidence.

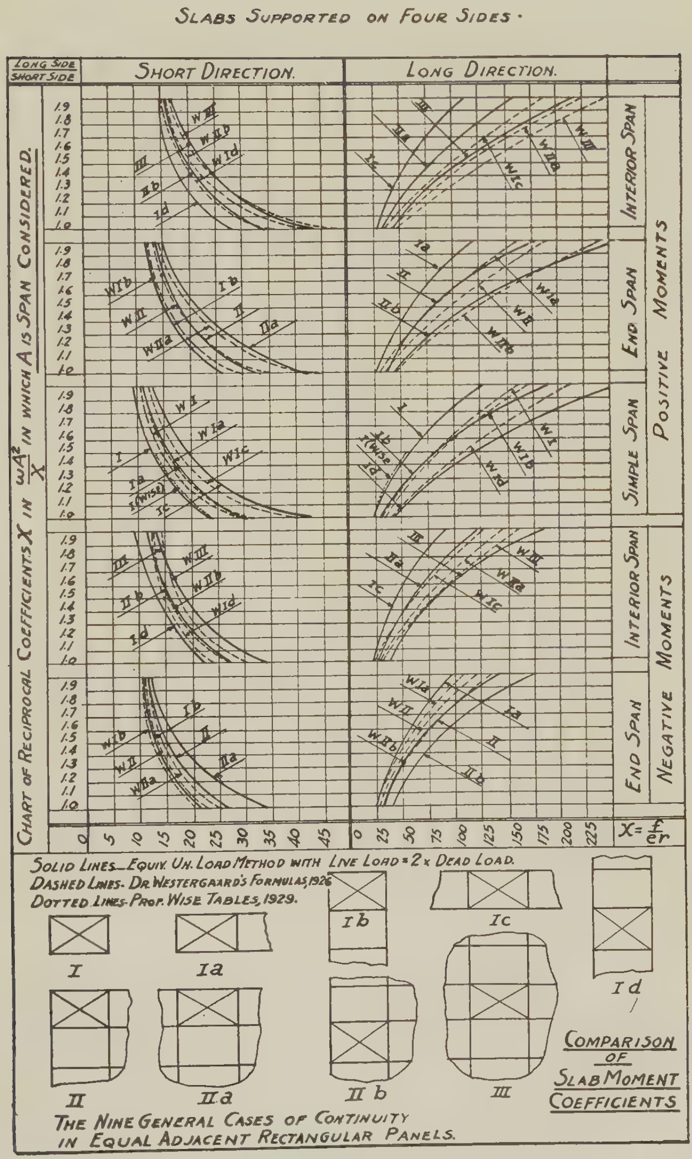
SLABS SUPPORTED ON FOUR SIDES.
LONG SIDE / SHORT SIDE
SHORT DIRECTION.
LONG DIRECTION.
CHART OF RECIPROCAL COEFFICIENTS X IN $\frac{wA^2}{X}$ IN WHICH A IS SPAN CONSIDERED.
POSITIVE MOMENTS
INTERIOR SPAN
END SPAN
SIMPLE SPAN
NEGATIVE MOMENTS
INTERIOR SPAN
END SPAN
$X = \frac{f}{er}$
SOLID LINES—EQUIV. UN. LOAD METHOD WITH LIVE LOAD = 2 x DEAD LOAD.
DASHED LINES- DR. WESTERGAARD'S FORMULAS, 1926
DOTTED LINES- PROF. WISE TABLES, 1929.
I
Ia
Ib
Ic
Id
II
IIa
IIb
III
THE NINE GENERAL CASES OF CONTINUITY IN EQUAL ADJACENT RECTANGULAR PANELS.
COMPARISON OF SLAB MOMENT COEFFICIENTS

FIG. 8

The effect of changing the location of the panel in the structure is indicated (Fig. 8) by the three curves in each diagram. It is clearly insufficient to consider one direction of the panel as a simple span, end span, or interior span without regard to the conditions of continuity in the direction at right angles. This would be even more apparent in the usual building structure where panels of irregular size, such as room and corridor, are encountered, but which are readily handled by the equivalent uniform load method.

FUTURE INVESTIGATION

Various factors influencing the behavior of two way slabs in some degree or in special situations offer fields for future research. These include the deflection of shallow girders, torsional resistance of the beams, dome action, Poisson's ratio, and the application of concentrated loads. While consideration has been given to these subjects in the course of the present studies, a general discussion of them is deferred pending further investigation. For usual conditions, these questions have minor significance.

CONCLUSIONS

In conclusion, it is deposed that the problem of slabs supported on four sides may be analysed logically and directly with satisfactory accuracy by this method. Each direction of the slab, under any condition of rectangularity, restraint, arrangement of the dead and live loading, or variation in the stiffness of adjoining bays, may be handled by one and the same formula by the assignment of proper values to readily determined factors. The design of the supporting beams proceeds rationally on the simple principle that the slab and parallel beams together carry the total load. Extensive studies show that designs carried out on this basis are economical and in line with good commercial practice. A simple and rapid solution of two way slabs as encountered in general building work is attained by means of equivalent uniform loads.

ACKNOWLEDGMENTS

Acknowledgments are gratefully made to A. W. Stephens, Chairman, Prof. Hale Sutherland, R. L. Bertin, and the other members of Committee 501, and to U. T. Berg for helpful suggestions and criticism.

REFERENCES

(1) Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders by H. M. Westergaard, *Proceedings*, A. C. I., 1926.

(2) Design of Reinf. Concrete Slabs by J. A. Wise, *Proceedings*, A. C. I., 1929.

(3) C. Bach, Versuche uber die Widerstandsfahikeit ebener Platten, Zeitchr. d. Ver. deutcher Ingenieure, Vol. 34, 1890.

(4) Previous codes limited two way slabs to those cases in which the long side is less than 1.5 times the short side. Recent investigations by Timoshenko indicate that this ratio may be as great as 3. However, when the ratio exceeds 2 the proportion of load carried by the long direction is very small and the ratio of effective depth to long span becomes excessive. Cases in which the panel ratio exceeds 2 should be treated as one way construction.

NOTATION

A is the span length between opposite supports in one direction.

B is span at right angles to A.

$F_A A$ is distance between lines of inflection in span A when span A only is loaded.

$F_B B$ is distance between lines of inflection in span B when span B only is loaded.

N is sum of lengths of those edges of panel A x B supporting continuous adjacent spans.

w is uniformly distributed load per unit area.

t_3 is minimum permissible slab thickness.

r_A is proportion of total load carried by span A of slab.

r_B is proportion of total load carried by span B of slab.

e_A is ratio of total equivalent uniform load influencing span A of slab to $r_A wA$.

e_B is ratio of total equivalent uniform load influencing span B of slab to $r_B wB$.

$\frac{1}{f}$ is bending moment coefficient.

K_A is stiffness factor $\frac{I}{A}$ for span A.

K_{AR} is stiffness factor for adjacent span.

Note: In the Committee's report, K_{AR} is used for stiffness of right adjacent span, and K_{AL} for stiffness of left adjacent span. For brevity in this article, K_{AR} is used for both, the adjacent spans being taken alike except as noted in text.

APPENDIX I

Derivation of F_A.

Let Fig. 6 represent an infinite series of spans A_R with a moment M applied at the left end. Let Q be the numerical ratio of moment at right end of any span to moment at left end of the same span. Then, from theorem of three moments

$$M - 4QM + QQM = 0$$

whence $Q = 2 \mp \sqrt{3} = .27$ ....(10)

In Fig. 7, given an interior loaded span A in an infinite series of unloaded spans A_R. Let I be moment of inertia of span A, I_R be moment of inertia of span A_R. Then from theorem of three moments

$$MAI_R + 2M\,(A_R I + AI_R) - QMA_R I = -\frac{wA^3 I_R}{4}$$

If K_A is $\frac{I}{A}$, K_{AR} is $\frac{I_R}{A_R}$, and M is $-\frac{wA^2}{f}$, then

$$K_{AR} + 2\,(K_A + K_{AR}) - QK_A = \frac{fK_{AR}}{4}, \text{ or}$$

$$f = 4\left(3 + 1.73\,\frac{K_A}{K_{AR}}\right) \qquad \text{(11)}$$

If $F_A A$ is distance between lines of inflection in span A,

$$\frac{F^2_A}{8} = \frac{1}{8} - \frac{1}{f}, \text{ or } F_A = \sqrt{1 - \frac{1}{1.5 - \frac{7K_A}{8K_{AR}}}} \qquad \text{(12)}$$

For an end span,

$$2\,(K_A + K_{AR}) - QK_A = \frac{fK_{AR}}{4}, \text{ or}$$

$$f = 4\left(2 + 1.73\,\frac{K_A}{K_{AR}}\right) \qquad \text{(13)}$$

and,

$$F_A = 1 - \frac{2}{f} = 1 - \frac{.25}{1 + \frac{7K_A}{8K_{AR}}} \qquad \text{(14)}$$

Discussion of this paper will be consolidated with that of the Report of Committee 501, transmitting proposed new building regulations and will be referred to Committee 501. See JOURNAL, *Nov.-Dec. 1935.*

Underwater Concrete Mixtures and Placement—San Francisco-Oakland Bay Bridge*

BY STANLEY M. HANDS†

TYPES OF FOUNDATIONS

The foundation concrete for the San Francisco-Oakland Bay Bridge was placed in salt water at depths varying from 25 to 242 ft. There were 29 large seals involving from 5,000 to 30,000 cubic yards of concrete for each seal and requiring continuous operations for more than a week at a time. The submarine bucket, lowered with crane and hoist was used to transfer the concrete from the mixer barges to the foundation at the greater depths. The tremie pipe was used in lesser depths. It may help to understand the conditions governing placing underwater concrete to describe briefly the means of reaching a satisfactory foundation material.

The penetration of the deep water and underlying mud was accomplished with caissons of two types, the domed cellular type for depths where additional caisson height was required for landing, and the false bottom type which could be landed on bottom without the need of building to a height that endangered stability and position in the tideflows. For each type, additional lowering and penetration of underlying strata was attained with the aid of the imposed weight as top construction was advanced and excavations were made through the cells or digging wells. When the cutting edges of these caissons reached adequate foundation materials the excavation was squared and cleaned to receive the concrete.

The plan dimensions of the caissons for Piers W3, W5, and W6 were 74.5 x 127 ft. and for the central anchorage W4 the plan was 92 x 197 ft. The excavation cells were 15 ft. in diameter. They were connected at the bottom through an adaptor section to the chamber enclosed by the cutting edge. The seal concrete was placed upon the

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foundation through these cells to fill the entire section to a point about 20 ft. above the top of the adaptor section.

Cofferdams were used in preparing foundations where good foundation material was available at depths which did not require the use of caissons or where it was possible to reach stable strata by means of piling. The cofferdams were constructed of steel sheet piling, stiffened on the outside by vertical I-beams driven to refusal and supported at the top by horizontal trusses, to give a single unobstructed working space within. Excavation of pier sites to —28 to —35 preceded erection of cofferdams. The cofferdams were then erected and excavation completed to —45 to —55.

W2 at —90 rested on rock. This cofferdam necessarily was framed inside. E2 at —45 rested on rock. For all other cofferdams 300 to 625 Douglas fir piling, varying from 80 to 110 ft. long, were driven to carry 30 to 60 tons load. The pile arrangement required straight and battered piles which were driven by subaqueous hammers with a follower where necessary. Excavation was seldom required around the tops of the piles.

Sand and gravel blankets, varying from 3 to 5 ft., were used to cover and bed down any bulked mud created by the driving operations and to insure a cleaner concrete in the seal. The seals were uniformly 16 ft. thick, allowing 12 ft. of concrete above the piling. There were 30 cofferdam seals in the major contracts, all of which were placed by means of the tremie.

AGGREGATE SOURCES

Because of the far-flung concrete operations on this bridge, it was necessary to insure an adequate supply of materials to meet any concreting requirements. To do this, aggregates were obtained from seven established producers. The normal products from these plants varied considerably, making it necessary to establish certain grading groups of definite range in sizes within which sufficient similarity could be maintained, so that materials from different sources could be used interchangeably without requiring important changes in the concrete mixture. When a change of source occurred the first barge (280 cu. yd.) to be batched did not always deliver exactly the desired type of mixture, but this condition did not cause delay, for a reasonable similarity between previous batches and those of the new materials could be secured with a slight adjustment of the water or some minor change in handling or placing such as filling the concrete receiving hopper before loading the bucket. It should be noted in passing that an essential feature of the success in handling this concrete was the

design of these concrete receiving hoppers and other concrete handling equipment. Further discussion of this will be found in the section on equipment.

TYPE OF CONCRETE REQUIRED

In selecting the type of mixture for these foundations, careful consideration was given to what we term "mass action," that is, the behavior of the concrete in the mass with respect to segregation, ability to flow, and tendency for self-consolidation. An important property of any concrete mixture is the ease with which it may be placed. In these structures the requirements were particularly severe. Not only was it necessary that the concrete fill all of the space within the excavation, but it had to flow through and around the pile tops without segregation, all of which had to be accomplished while placing it through the water. Because segregation and infiltration of water could not be observed, under water concrete had to be designed so that nothing was lost from or added to the mass by reason of its flow. In other words, the mixture had to be self-fabricating. It was the selection of the mixture to accomplish these ends rather than economy in cement that we emphasized as of primary importance among the problems connected with the placing of concrete under water

Laboratory tests by T. E. Stanton and the writer indicated that for materials from different sources the behavior in the mass was not always similar for the same grading and proportions, but that combinations could be found for materials from most sources which would give much the same type of concrete mixture. This was fortunate, for the gravel deposits within shipping distance of the work contained only 15 per cent of sizes greater than 1½ in. Since the concrete usually contained more than 25 per cent larger than 1½ in. and 35 per cent greater than the 1-in., most of the larger sizes came from adjacent quarries. Also, on the basis of the laboratory tests, it was decided to use a plastic, not fluid, mixture which contained about 2 per cent air. Mixes that were more fluid contained less air, but were less cohesive; drier mixes contained more air. Excess air was believed to be especially undesirable in under water work, for while the air lends something to the flow of the concrete, it seemed likely that its escape under water would be accompanied by undesired turbulence. In the type of mix selected the sheen of free water showed during vibration, but disappeared when vibration ceased.

IMPORTANCE OF FINES

Special attention was given to the proportions of fines in the sand. More and more the importance of this size range is being recognized

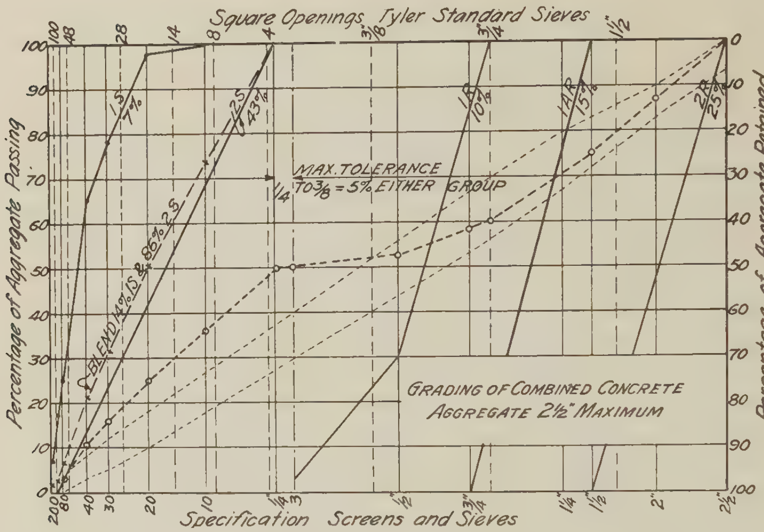

FIG. 1—INSPECTION REPORT (SEE ALSO FIG. 2)

and producers of aggregate are being worried about the demand for control of the finer sand fractions. In California, the practice is to use 30-mesh screen as the point of control for the finer fractions. Some other size might be just as good, but if the full effect of the finer materials is to be recognized and controlled there should be more screens for the finer sizes than are found in the standard set. Practically, the effect on the mixtures of relatively small amounts of these finer sizes is very significant.

Sand was separated and reblended in hydroseparators. Limited groups of the smaller sizes and a basic sand having a uniform coarser grading were pumped in fixed amounts to a classifier where they were mixed under water. This method was copied from Boulder Canyon operations and is reported in the August, 1935 issue of *Rock Products.* To insure against delivery of any materials other than those agreed upon as desirable and practically producible, inspection and identification was made at the source. As a matter of policy it was made clear to the producers that acceptance was not to be inferred because we elected to identify the materials at the source. This testing, reports of which were delivered with the shipments, was made to enable train crews to block out similar materials and to permit the batching plant inspector to anticipate the deliveries and the need for changes in the proportions which must be made when materials changed. Some

such method is necessary when quantities reach several hundred cars daily.

Attempts to load and blend with mixing belts or by alternately loading different size groups in layers in the cars or on the barges were found to be unsuccessful in providing a uniform grading. Incidentally, bunker segregation of the coarse aggregates at the plant had little effect on the behavior of under water mixes. This reflects the advantage of separating into limited size groups and the use of excess mortar for this particular type of work.

In addition to the emphasis given this finer fraction, control was made effective by limited tolerances on all the groups up to 1 in. With effective control over the intermediate sizes and with the right kind of mortar, either crushed rock or gravel for the larger materials could be used with little or no difference on the action of the mixture.

PROPORTIONS

Typical underwater concrete was designed with 50 per cent of sand, 6 sacks of cement per cubic yard and a water-cement ratio by volume of 0.85. By way of comparison, structural concrete mixtures were designed with 37 per cent of the same sand, 5 sacks of cement (for heavy construction) and a water-cement ratio of 0.85. Thus, the underwater mixture contained 13 per cent more mortar than the regular structural mixes. The ease with which the underwater mix was placed and the uniform quality of concrete as determined from field samples and cores justified the use of this extra cement. The average compressive strength of 497 field cylinders in 28 days was 3721 p.s.i. Cores drilled through the seals showed uniform fabrication and density, and specimens made from these cores gave approximately 5000 p.s.i. at 1 year.

The usual practice was to fix the combined grading from the trials and thereafter the batching plant inspector calculated the proportions of materials to maintain this combined mixture. This is a simple method for permitting the use of materials which depart somewhat from the specified gradings. Sample inspection reports are shown in Fig. 1 and 2. A tolerance in grading limits of five per cent was allowed. This is not an unreasonably close tolerance and was fixed by the producers as adequate. Except in the finer fractions greater ranges were observed in testing, but no effect upon the mix was observed when these materials were used without changes in the proportions. The difference probably was due to relatively small changes in shape or size which made the quantities above and below the larger screens vary.

INSPECTION—FORM 1

Size	2R 25%	1AR 15%	1R 10%	2S 50%	Combined Mix
2½	100				100
2	48				87
1½	0	100			75
1		40			56
¾		0			60
			95		60
½			30		52
3			0	100	50
10				73	36
20				50	25
30				32	16
40				21	10
80				6	3
100				2	1
200					
Source	Ant	"c"	"c"	"c"	

$$G = \frac{1}{\frac{*.07}{2.56} + \frac{*.43}{2.66} + \frac{.10}{2.69} + \frac{.40}{2.71}} = \frac{1}{.3737} = 2.675$$

5 Sack Structural W/C = .85

K = 71.225 cu. ft. =

	%	Lbs.	Cu. Ft.
Cmt.			
2S			
1R			
1AR			
2R			
H_2O			
Total			

Mixture 1 : . . . . . . : . . . . . .

Sp. Grav. Cement 3 : 14

6 Sack Underwater W/C = .85

K = 66.57 cu. ft. W = 11,130 lbs.

	%	Lbs.	Cu. Ft.
Cmt.		1974.00	10.08
2S*	.07 .43	779.10 4785.90	4.85 28.80
1R	.10	1113.00	6.66
1AR	.15	1669.50	9.84
2R	.25	2782.50	16.42
H_2O		1115.62	17.85
Total			94.50

Mixture 1 : . . . . . . : . . . . . .

*Blend 14% hydro and 86% std.
Sp. Gr. 2.56 and 2.66 resp.
K = 94.5 — 6 x 3.5 (.85 + .48) = 66.57 cu. ft. Abs. Vol. of aggregate
W = Weight of batch for above parts 66.57 x 2.675 x 62.5 = 11,130 lbs.

LLK.

Mix No. 256

FIG. 2 (SEE ALSO FIG. 1—SAMPLE INSPECTION REPORTS)

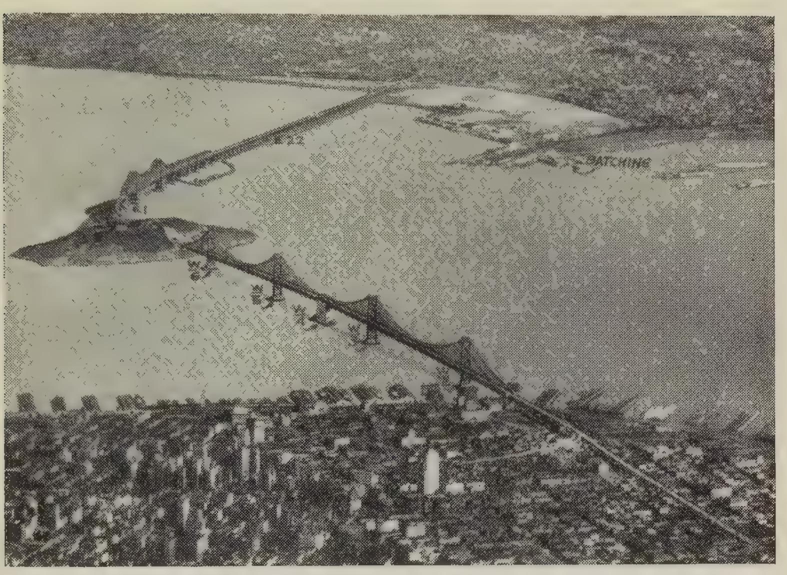

FIG. 3—UPON AN AERIAL PHOTOGRAPH OF SAN FRANCISCO BAY THE ARCHITECTS MADE A SCALE REPRESENTATION OF THE BRIDGE. THIS WITH THE MARKS ON THE PICTURE GIVE A GENERAL IDEA OF THE JOB

There was practically no ¼ to ⅜-in. material in the aggregate. This size seems to be the cause of trouble when the mixes are loaded with the larger sizes. Whenever there appeared to be too much big rock in the mix we took out some of the middle and added it to the finest and coarsest materials. This made more big rock in the mix but it would disappear within the mortar. This is an example of particle interference and a correction.

EQUIPMENT

In addition to the type of mixture one of the important factors in underwater practice is the rate at which the concrete is placed. It is desirable that the mass of live concrete be as great as possible in order that the weight help to weld the mass together. Therefore, the selection of adequate equipment was reserved for approval by the engineer. Plants furnishing materials, transportation facilities, batching plants, handling methods, tow boat power and navigation were all investigated in an attempt to foresee any possibility of a delay in seal operations. Control over these items is implied from a clause in the specifications which provided that seal operations should not begin until an adequate supply of concrete materials was assured at the site of operations.

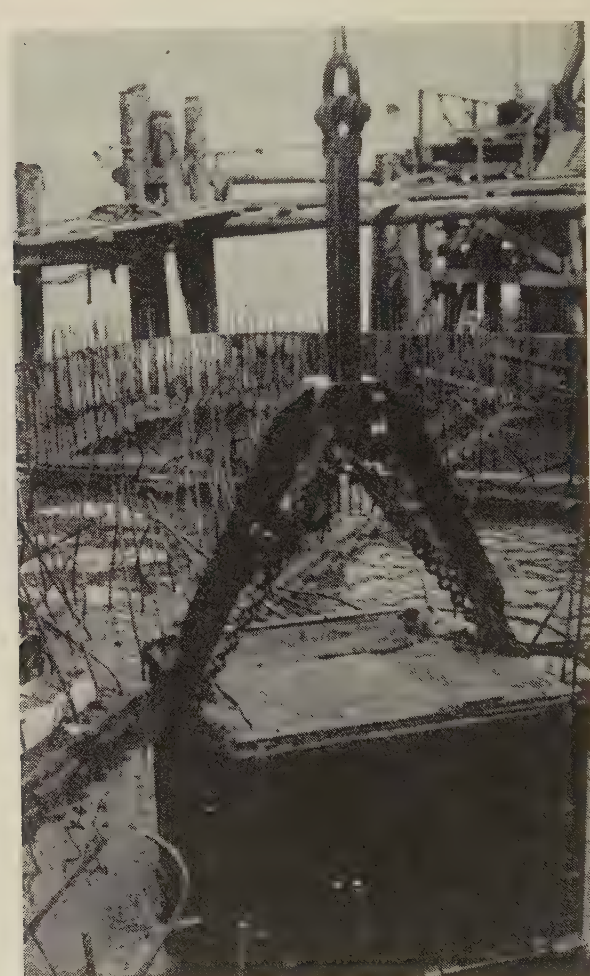

FIG. 4—BUCKET OF CONCRETE, WITH A CANVAS COVER, FOR PLACING UNDER WATER

The concrete mixtures for marine work were batched with auto-scale equipment having a theoretical capacity of 250 cu. yd. of concrete per hour. The batching plant was on the Southern Pacific wharf in Oakland and therefore was served by rail and water facilities. The mixer barges were equipped with four 3.5-cu. yd. mixers which were charged and discharged with belt conveyors. Since the discharge belt was on an incline it is apparent that underwater mixtures could not be liquid mixtures. Each barge carried 280 cu. yd. of dry batches to which the water was added at the pier site when mixing operations

FIG. 5—PLACING FIRST SECTION STEEL FORMS FOR PIER E-3

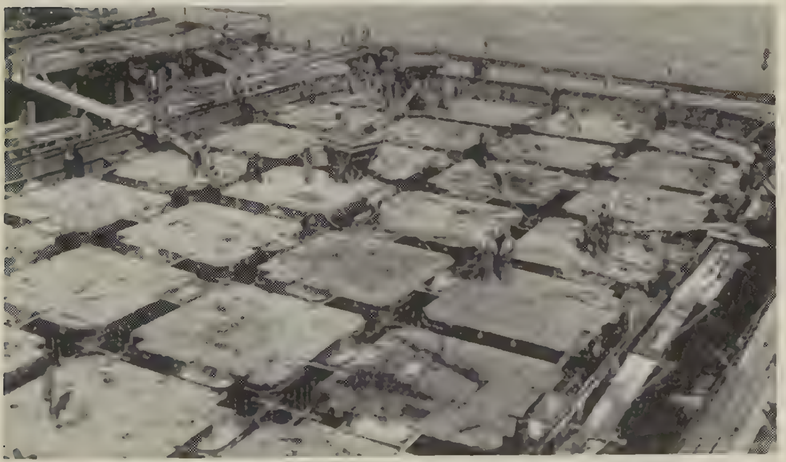

FIG. 6—DIGGING WELLS COVERED WHILE BUILDING UP—NOTE TRAYS FOR CROSS-WALL PLACING WITH VIBRATOR CONCRETE

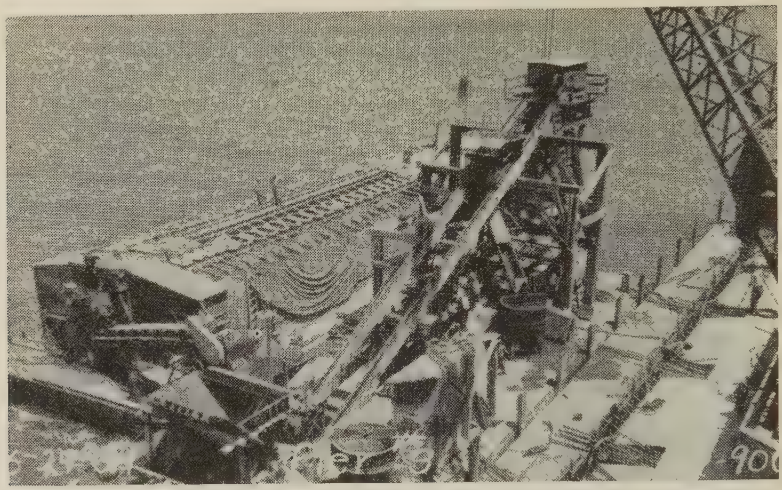

FIG. 7—CONCRETE TRANSFER BARGE, BETWEEN MIXER BARGE AND CAISSON

were under way. Moisture tests were made for batching the aggregates and because of the loss of moisture in transit another test was run prior to mixing to fix the additional mixer water.

Navigation during the heavy fogs was safely carried out even though all loads crossed the ferry lanes. This type of batching plant and mixer barge was selected because of the greater investment which would be required in barges if separate materials were delivered to the site. The transfer of full and empty barges under the tide conditions within the limited work areas around the pier sites would have been expensive if practical.

An important part of the several types of equipment which were devised for this work was the concrete hoppers and skips. These were all devised to give the concrete batches a mixing action as they were discharged. The sides were battered and the discharge gates were located in the bottom, or partially in the bottom. The batches from the mixers were delivered into receiving hoppers before they were discharged into skips or buckets. This practice corrected any segregation by the belt conveyor.

CONTROL OF PLACING

Since the mixers were locked against discharge during the specified period of mixing, the practice of putting the full batch into the receiving hopper before discharging largely controlled the rate of placing. The crane operators raced to keep the hopper empty. This required

FIG. 8—UNDERWATER CONCRETE DELIVERED BY BELT TO BOTTOM-DUMP RECEIVING HOPPER, THENCE TO BOTTOM-DUMP CAR, UP INCLINE TO UNDERWATER PLACING BUCKET

that the inspector "bear down" frequently to be sure that the bucket was lowered gently into the water until submerged. Otherwise the canvas flap covering the concrete might be thrown out of position by the eddy currents and the surface of the concrete exposed to the wash as it was lowered into place. For tremie pipe practice this was not a factor, but there was a very marked tendency to lift the pipe so that the flow was continuous from the gantry hopper to the seal. After a few pipe seals were lost by this practice the hoist operator on the gantry damped the flow and allowed the pipe to fill slightly as the concrete was fed downward to the seal. This damper effect is produced by lowering the end of the pipe into the live seal concrete so that the back pressure will stop or slow the flow.

In sealing the cofferdam three tremie pipes were used together. They were supported in position by a gantry frame mounted on the same tracks on the top of the same frame as was used by the pile driver. Within the gantry frame three motor driven hoists were rigged to give separate control of the tremie pipe for damping and flow. Above this gantry and mounted on rails fixed to the top chord of the gantry frame a traveling hopper moved back and forth to discharge concrete into the tops of the tremie pipes. This traveling hopper was supplied by a whirley crane and bucket with concrete from the mixer barges.

Intermittently during the pour, inspectors sounded with a weighted graduated wire for depth and position of the concrete. The State diver made inspections to confirm surface tests or soundings and with this check inspectors developed means of correlating the action of the concrete above and below water.

The hazards of stopping operations were recognized and if conditions or events indicated that uniform results were not attained, placing operations were carried out cautiously during the time required to locate the cause and fix the remedy. As an example, if the usual mechanical troubles in batching plant operations delayed deliveries to the site notice was given immediately by short wave radio that the next barge would not arrive on schedule, therefore the rate of placing tremie concrete was cut down to insure that live concrete would be in the seal when the next barge arrived.

Direct radio and telephone communication facilities were maintained between the state and contractors headquarters and pier sites as well as with concrete barges and tow boats. The engineers had a different wave length to that used by the contractors.

SUMMARY

A summary of our experience shows that we have emphasized the following points for placing concrete under water: They are all equally important and bear directly on any degree of success which we have attained:

Prequalification of inspection personnel by means of reviews of appropriate literature together with occasional reports covering details of assignments especially as related to new problems and the means of solving them.

Prequalification of equipment and methods as well as certain labor classifications to insure adequate quality and quantities.

A division of aggregates into size groups, of such limits as to be readily produced by a variety of plants.

Requiring that each size group be furnished within 5 per cent of the specified size limits.

Those mixtures which could have been used to economize on the cement were rejected in favor of the mixtures which provided greater assurance of uniform fabrication under water.

The work upon this project was carried on under the direction of C. H. Purcell, Chief Engineer, Charles E. Andrews, Bridge Engineer, and Glenn B. Woodruff, Engineer of Design. Preliminary studies of concrete materials and concrete as related to the preparation of concrete specifications were made under the direction of T. E. Stanton, Materials and Research Engineer, Division of Highways, State of California. Design and control of concrete mixtures was under supervision of the concrete staff of the Materials and Research Laboratory. Concrete placements were made as directed by V. A. Endersby, Resident Engineer, Contracts 4, 4A, and 8, and I. O. Johlstrom, Resident Engineer, Contract 2. These contracts covered all work involving underwater concrete.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1936. Discussion should reach the Secretary by April 1, 1936.*

Discussion of a Paper by Messrs. Ruettgers, Vidal and Wing:

"An Investigation of the Permeability of Mass Concrete with Particular Reference to Boulder Dam"*

AUTHORS' CLOSURE

The authors are appreciative of the interesting and helpful discussions submitted by Messrs. Mary, Carlson, Norton and Pletta, and Meyers. The theoretical discussion of the chemical reactions which take place as water percolates through concrete and of the differences likely to be caused by waters of varying composition helps materially to round out the subject.

The original paper was based on tests of 138 concrete specimens. Since its presentation the results of about double this number of tests have become available for analysis. In addition the writers have had access to the excellent reports on the same subject by Mr. Mary†, whose test equipment is similar to that used by the Bureau of Reclamation but whose test specimens were made with higher water-cement ratios. A study of the present data has confirmed, in general, the conclusions in the original paper. Increased use of the permeability coefficient has demonstrated its utility for reducing test data on specimens of different sizes to a common basis, for comparing the work of different experimenters, and for estimating for actual structures the adequacy of a given mix with respect to percolation. It is not claimed that the coefficients are perfect indices of relative permeability or that by their use accurate forecasts of flow through field structures can be made. The experimental coefficients, even with the exercise of the greatest care in testing, are disappointingly variable and the true influence of such factors as time and skin effect can only be obtained by accumulating more data. However, even if an estimate of the expected leakage through the internal pores of a field concrete can

*Journal, Amer. Concrete Inst., Mar.-Apr. 1935; *Proceedings*, Vol. 31, p. 382; for discussion see Sept.-Oct. and Nov.-Dec. Journal 1935, *Proceedings* this Volume, p. 125 and 230.

†Annales des Ponts et Chaussees (Mar.-Apr. 1933 and Nov.-Dec. 1934). Reviewed by B. Moreell, Journal, Amer. Concrete Inst., May-June 1935; *Proceedings*, Vol. 31, p. 571.

be made within several hundred per cent it is better than no estimate, and for those cases for which chemical deterioration by percolating water is likely to be a controlling factor this uncertainty, great as it is, may be cared for by a relatively small change in the mix.

The original paper indicated that inasmuch as the usual aggregate can be considered impermeable as compared to the paste, percolating water may be conceived as flowing alternately through the pores between cement grains and the much larger interstices formed by the settlement of the paste below aggregate particles. While the tightness of paste made of commercial cement is probably largely controlled by the water-cement ratio, the characteristics of the cement with respect to fineness and chemical composition are certain to be modifying influences as implied by Mr. Meyers. Unfortunately the permeability tests of the Bureau covering these characteristics were too few to yield conclusive results, and the only permissible inference is that the differences in the cements used were of less importance than the water-cement ratio. The other main factor controlling the rate of percolation, namely the formation of large interstices below aggregate particles, seems to be a function of the grading and size of the aggregate and of the settlement characteristics of the paste.

Mr. Carlson's microscopic work explains why the hydration of the cement, though causing but a small decrease in the void space, results in a reduction of the permeability of the order of ten thousand times. The colloidal gel with perhaps a porosity of 90 per cent is, according to Mr. Carlson, practically impermeable. Of interest in this connection is a recent test made by the Bureau.

A 6-in. diameter concrete specimen, 2 in. thick, of 1:3.5:3.5 mix, with ¾-in. maximum aggregate and 0.70 water-cement ratio, was subjected to the usual percolation test at 200 pounds pressure giving an uncorrected value of $K_c = 134 \times 10^{-12}$. At the close of the test salt water of known concentration was substituted for city water and the percolation test was continued until by analysis salt water of substantially constant concentration came through the specimen. Frequent analyses of the issuing water were made, from which three useful facts were obtained. First, the time at which salt initially appeared in the filtrate permitted a computation of the velocity of the water passing through the largest pore; second, the time at which the issuing water was of about the same salt concentration as the initial supply gave an approximation of the velocity of the smallest pore; and third, the total amount of salt solution absorbed by the specimen was a direct measure of the total pore space. The computed velocity in

the largest pore was 8 ft. per day, the nominal mean velocity $\frac{Q}{A \times \% \text{ voids}}$ was 0.15 ft. per day, and the velocity in the smallest pore 0.016 ft. per day. The absorbed salt solution was 19.8 per cent by volume of the specimen or 97 per cent of the volume of the free water (added) at the time of the mix. Thus the minimum, nominal mean, and maximum velocities through the pores were in the relation of 1:10:500. As it is to be expected that the greater portion of the measured discharge comes from the larger pores it is indicated that most of the water flows at a rate of 40 to 50 times that computed by using the coefficient K_v.

Various experimenters studying flow through granular materials have estimated that the velocity of flow through irregular pores is from 1/20 to 1/100 of the velocity in a uniform tube of equivalent volume. On this basis the equation for the velocity of flow through a pore in concrete is $V = 2000\frac{H}{L}d^2$, in which "*d*" is the diameter in feet of the equivalent uniform pore. Substituting in this equation the measured velocities and known hydraulic gradient gives the dimension of the smallest and largest equivalent pore as 0.05 and 1.2 microns respectively. Less than 10 per cent by weight of the cement is smaller than the latter dimension. These quantitative dimensions though subject to large error probably give at least the order of size of the pores in the concrete specimen tested. This is a matter of interest in considering alternative methods for improving the watertightness of concrete.

This single test is presented mainly with a view to its future possibilities. If instead of a diffusing salt solution an immiscible fluid such as gasoline were to be used, such a fluid would displace only water in the active pores. It would then be possible from the analysis of the filtrate to obtain data more characteristic of the pore distribution. A better knowledge of the fundamental makeup of the internal structure of concrete to which this type of test should contribute might explain many apparent anomalies to existing data.

From the discussions of Messrs. Norton and Pletta, and Mr. Mary it appears that some clarification of the purpose, experimental methods and theory of percolation tests as carried out by the Bureau is required. The Bureau's tests were made for the definite purpose of estimating the effect of percolating water on the concrete of Boulder Dam over a long period of years. As any effect produced would be related to the total amount of water which had percolated, the first requirement was

an estimate of an average rate of flow which would be applicable for an indefinite period. The rate of flow through a pore, as in a pipe, is governed by its cross-sectional area and by the character of the pore surface. It seemed, therefore, that for a test to be of value it not only would have to be made under conditions of more or less complete hydration of the cement but also that the specimens would have to be made from the actual proposed mix. Otherwise the pores tested would bear no resemblance to those for which information was required. Moreover, even for relative results, theory seemed to indicate that comparisons made of rates obtained early in the test before the pores were completely filled with water, might be no more characteristic of the relative rates which would obtain during the life of the concrete than would rates of filling reservoirs be characteristic of the rate of passage of water through the reservoirs when full. For these reasons the writers disagree with Mr. Mary's proposal to compare rate curves as a whole in which the initial rates obtained during the filling of the pores are combined with the rates after steady flow is established. The same objection is applicable to Messrs. Norton and Pletta's method of using the 40 to 50 hour rates for comparing 6-in. specimens tested at 40 to 100 pounds pressure.

The absorption curve of Fig. 2 in the original paper, derived from the difference between the measured inflow and outflow from a specimen under 400 pounds pressure shows that for this 6-in. specimen the absorption continued for some 300 hours. At 100 pounds pressure, as used by Norton and Pletta, 1200 hours would have been required to produce the same saturation. The space into which this water was absorbed was largely created by the hydration of the cement. Had some of the free water been removed from the pores by previous exposure to normal room humidity and temperature the absorption would have been materially greater and a longer time would have been required for saturation. It seems certain that rates taken as early as 40 or 50 hours at comparatively low test pressures will be greatly influenced not only by the initial type of curing but also by the extent to which the pores happen to be saturated with water, a characteristic varying greatly with small changes in humidity and procedure.

Except in special cases the tests of the Bureau were generally made on 2 to 4-in. slump concretes with workabilities eminently suitable for field use. Cylinders were cast using a standard procedure for compaction except that very large pieces of aggregate were hand-placed to insure uniform distribution. Compressed oxygen cylinders were used to supply pressure to the water reservoirs. The permeability coefficient was derived first by dividing the measured rate of

TABLE 3—AVERAGE PROPERTIES OF PERMEABILITY SPECIMENS, AND TEST RESULTS

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Size of Cylinder in Inches—Diameter by Length	No. Specimens Averaged	Mix by Weight C:S:G	Maximum Size Aggregate, In.	Water-Cement Ratio by Weight	Slump in Inches	Curing Method — S = Standard	Initial Curing Temperature °F	Age at Start of Test in Days	Time on Test Before Outflow, Days	Perm. Coefficient (K'_c) Age, Days	Pressure Head in Feet of Water	Pressure Head in lb. per sq. in.	Age When Broken, Days
EFFECT OF WATER-CEMENT RATIO WITH NEARLY EQUAL PASTE CONTENTS													
6 by 12	4	1:1.95:3.46	1½	0.30	0	S	70	28	0	28	231	100	97
6 by 12	4	1:2.26:4.01	1½	0.40	0	S	70	28	—	42	923	400	160
6 by 12	3	1:2.42:4.29	1½	0.45	½	S	70	28	—	45	923	400	83
6 by 12	4	1:2.57:4.55	1½	0.50	⅛	S	70	35	—	49	923	400	.....
6 by 12	3	1:2.73:4.83	1½	0.55	1	S	70	28	—	39	923	400	.....
6 by 12	14	1:3.06:5.43	1½	0.66	4¼	S	70	82	3	93	923	400	.....
6 by 12	4	1:3.35:5.94	1½	0.75	7	S	70	38	1	49	923	400	.....
6 by 12	3	1:4.13:7.32	1½	1.00	8½	S	70	28	—	38	231	100	131
EFFECT OF MAXIMUM SIZE OF AGGREGATE													
6 by 12	2	1:3.06:4.50	¾	0.66	3¼	S	70	54	2	65	923	400	.....
6 by 12	14	1:3.06:5.43	1½	0.66	4¼	S	70	82	2	93	923	400	.....
8 by 8	3	1:3.06:6.62	3	0.66	4¼	S	70	54	2	65	923	400	.....
12 by 12	5	1:3.06:8.08	4½	0.66	3¼	S	70	50	2	61	923	400	.....
18 by 12	4	1:3.06:8.08	4½	0.66	3¾	S	70	30	10	45	923	400	.....
EFFECT OF WATER-CEMENT RATIO ON NEAT CEMENT SPECIMENS													
6 by 2	3	Neat cement	.....	0.60	.....	S	70	28	1	39	231	100	.....
6 by 2	3	Neat cement	.....	0.80	.....	S	70	28	1	39	231	100	.....
6 by 2	3	Neat cement	.....	1.00	.....	S	70	28	1	39	231	100	.....
6 by 2	3	Neat cement	.....	1.20	.....	S	70	28	1	39	231	100	.....
EFFECT OF TYPE OF COMPACTION													
6 by 12	14	1:3.06:5.43	1½	0.66	4¼	S	70	82	2	93	923	400	.....
6 by 12	4	1:3.06:5.43	1½	0.66	4	S	70	29	5	42	923	400	.....
END EFFECT OF SPECIMENS													
6 by 6	5	1:3.06-5.43	1½	0.66	4¾	S	70	58	2	69	923	400	.....
6 by 12	14	1:3.06-5.43	1½	0.66	4¼	S	70	54	2	65	923	400	.....
6 by 18	3	1:3.06:5.43	1½	0.66	5	S	70	52	2	63	923	400	.....
6 by 24	3	1:3.06:5.43	1½	0.66	4½	S	70	53	3	65	923	400	.....
EFFECT OF DIRECTION OF PRESSURE APPLICATION													
6 by 12	3	1:4.04	No. 4	0.67	3	S	70	159	15	174	923	400	.....
6 by 12	3	1:4.04	No. 4	0.67	3	S	70	84	30	110	923	400	.....
6 by 12	14	1:3.06:5.43	1½	0.66	4¼	S	70	82	2	93	923	400	.....
6 by 12	6	1:3.06:5.43	1½	0.66	3½	S	70	23	0	34	923	400	.....
12 by 12	5	1:3.06:8.08	4½	0.66	3¼	S	70	50	2	61	923	400	.....
12 by 12	4	1:3.06:8.08	4½	0.66	2¾	S	70	28	2	37	231	100	.....

Notes:

General: All specimens were fog cured at 70° F.
Yosemite Special cement and Boulder Dam aggregate were used in all specimens.

TABLE 3—(Continued from Page 382)

15	16	17	18	19	20	21	22	23	24	25	26	27	28
Unit Breaking Strength, lb. per sq. in.	Weight of Specimen in lb. per cu. ft.	Apparent Loose Volume of Agg. in conc., % by vol.	Porosity of Aggregate, % by Volume	Cement Content, bbl. per cu. yd.	% Paste (cem. + water + air) in conc. by Absolute Vol., "P"	% Water Voids in Concrete by Volume, "V"	% Air Voids by Volume in Concrete	$\frac{\text{Water Voids}}{\text{Cement + Water}}$ by Abs. Vol.	K'_c x 10^{12} = Coefficient of Permeability in ft. per sec. at Age Shown in Column 11	K_c x 10^{12} = Coefficient Adjusted to 60 Days Curing and 12 in. Length	K_p x 10^{12} = Coefficient with Respect to Paste = $\frac{K_c}{\%P}$	K_v x 10^{12} = Coefficient with Respect to Voids = $\frac{K_c}{\%V}$	See Notes
EFFECT OF WATER-CEMENT RATIO WITH NEARLY EQUAL PASTE CONTENTS													
2,750	....	.....	.....	.....	.....	.....	.....	0.482	....	....	120,000	....	†
5,100	151.8	95.9	22.3	1.42	25.5	12.7	2.6	0.556	3	2	8	16	
4,070	149.9	95.3	22.3	1.32	26.0	13.3	3.2	0.583	3	3	12	23	
....	150.5	96.4	22.3	1.24	25.2	13.8	2.4	0.610	6	5	20	36	
....	152.9	98.1	22.3	1.20	22.8	14.8	0.4	0.632	4	3	13	20	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	85	75	310	480	
....	152.5	99.1	22.3	0.99	23.0	16.6	0.0	0.700	110	80	350	480	
1,200	151.6	99.7	22.3	0.81	22.5	18.1	0.0	0.757	7,110	4,670	20,700	25,800	
EFFECT OF MAXIMUM SIZE OF AGGREGATE													
....	149.4	97.4	24.2	1.16	26.2	17.4	0.6	0.672	14	14	55	80	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	85	75	310	480	
....	151.8	96.0	20.0	1.00	23.3	14.7	1.8	0.672	75	90	390	610	
....	154.0	98.1	18.2	0.86	19.9	12.7	0.9	0.672	240	220	1,100	1,730	
....	153.3	97.7	18.2	0.86	20.1	12.7	1.2	0.672	220	160	800	1,260	‡
EFFECT OF WATER-CEMENT RATIO ON NEAT CEMENT SPECIMENS													
....	....	.....	.....	.....	100.0	65.1	.....	....	....	....	100	150	
....	103.0	.....	.....	.....	100.0	71.3	.....	....	....	....	1,830	2,570	
....	95.9	.....	.....	.....	100.0	75.6	.....	....	....	....	3,730	4,930	
....	....	.....	.....	.....	100.0	78.9	.....	....	....	....	8,530	10,800	
EFFECT OF TYPE OF COMPACTION													°
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	85	75	310	480	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	21	14	60	90	**
END EFFECT OF SPECIMENS													
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	65	70	290	450	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	85	75	310	480	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	190	200	830	1,270	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	170	180	740	1,150	
EFFECT OF DIRECTION OF PRESSURE APPLICATION													
....	136.5	88.1	33.5	1.72	41.4	25.6	3.3	0.676	14	23	55	90	
....	136.5	88.1	33.5	1.72	41.4	25.6	3.3	0.676	14	20	50	80	††
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	84	70	290	450	
....	151.0	97.6	22.3	1.07	24.2	15.7	0.8	0.672	150	80	330	510	‡‡
....	154.0	98.1	18.2	0.86	19.9	12.7	0.9	0.672	240	220	1,100	1,730	
....	154.0	98.1	18.2	0.86	19.9	12.7	0.9	0.672	330	190	960	1,500	‡‡

†K_p was estimated from a one-hour test.
°Valves of K_c for this group have been corrected for effect of age only.
‡The individual valves of K_c were very erratic—540, 100, 7 and 4.
**Specimens were compacted by vibration.
††Specimens were tested upside down.
‡‡Water pressure was applied at right angles to the direction of casting.

discharge at 250 hours plus one-half the time to visible outflow, by the gross area of the specimen and then reducing the rate to its equivalent at unit hydraulic gradient. This value was reduced, if necessary, to a value equivalent to 60 days of total cure and to a result which would have been obtained had a 12-in. specimen been used, by means of the correction curves presented in the original paper. The value thus obtained is called K_c. The coefficient K_p was obtained by dividing K_c by the per cent volume of paste in the mix. Likewise K_v equaled K_c divided by the per cent by volume of original water in the mix as this volume of water was found to approximate the total pore space in the cured specimen. K_v is a direct measure of the mean nominal velocity of the water in the pores and is considered to be the significant coefficient for the purpose of comparing the probable relative resistances of concretes to deterioration by percolating water. Later in this discussion experimental data are presented showing that at least in comparing neat cements and concretes complete reversals in the order of permeability are found depending on whether the specimens are arranged according to K_c or K_v.

It should be emphasized that while a concrete may have a minimum permeability coefficient K_v as defined, it may at the same time be less suitable than another concrete of equal or higher permeability for resisting the deterioration caused by water moving through the pores by capillarity and/or by freezing, inasmuch as the factors controlling capillary flow are not the same as those for flow under pressure. Failure near foundations or at water level due to capillary water is a common phenomenon yet, as far as the writers know, few suitable tests have been made to define this characteristic.

Though not directly applicable to an author's closure of a paper it is thought that presentation of the more important of the additional data obtained since the original paper was published will be welcome. Table 3 tabulates the average physical properties of the specimens and the derived permeability coefficients for these tests. Some idea of the difficulties of the experimental technique may be obtained by examining the computed absolute volumes of paste as given in Column 20 under "Effect of Water-Cement Ratio with Nearly Equal Paste Contents." In this series constant absolute volumes of paste and aggregates were used, and as the aggregate structure is the same for all specimens, the tests show the effect on permeability of variation in the ratio of the absolute volume of water to cement in the pores between the aggregate grains. In spite of great care, entrained air (which is considered a part of the paste) caused the volume of the paste to vary from 22.5 per cent to 26 per cent of the volume of the

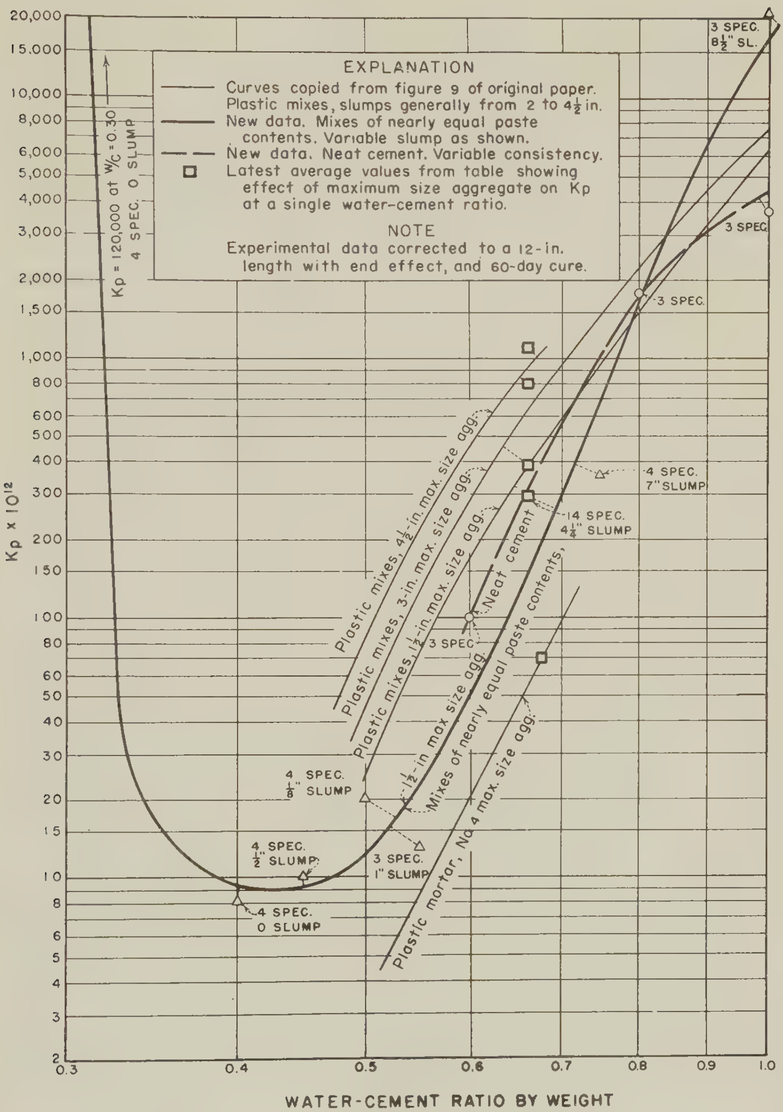

FIG. 14—ADDITIONAL DATA ON PERMEABILITY COEFFICIENT OF PASTE

mix, a 16 per cent variation. The permeability coefficients found for this series are plotted in Fig. 14. This curve of coefficients for 1½-in. maximum aggregate mixes is of theoretical rather than practical interest, for with the slump varying from 0 to 8½ in. the mixes are not uniformly workable. Differing as the curve does from the earlier

1½-in. series of Fig. 9, (reproduced in Fig. 14) it indicates that the proportions, grading and consistency of a mix may be factors affecting permeability.

Additional tests were made for the purpose of clearly showing the effect of maximum size of aggregate. As shown in the table the ratio of paste to sand was kept constant but the quantity of coarse aggregate and its maximum size was varied in such a manner that the slump was approximately uniform. The coefficients from this series of tests are also plotted on Fig. 14. It can be seen that in general they check the points given by the previous curves though the departure of an individual value from a specific curve is large.

One of the most interesting of the recent tests is a series made on neat cement with varying water-cement ratios, the results of which are likewise plotted on Fig. 14. Contrary to the commonly stated view that neat cement is practically impermeable, the K_p values indicate that at any given water-cement ratio the neat paste is more permeable than the series made with 1½-in. maximum aggregate and constant paste content and much more permeable than mortar. Moreover if it be compared on the basis of leakage of water per unit of gross area (K_c) it will be found that the neat cement specimens are the most permeable of all the mixes tested. With this characteristic of neat cement in mind it can be seen that the design of a mix for optimum impermeability consists in the addition of properly graded aggregate to a given paste in an amount sufficient to break up the direct passages of water through it to the greatest possible extent, but at the same time in the avoidance of a quantity or grading of the aggregate of a type tending to the formation of pores by paste settlement. That neat cement is so permeable may at first seem unreasonable to those who have depended upon it for watertight facings and the like. In such cases however the use of a much lower water-cement ratio in the paste than is practicable for the concrete on which it is placed may make it the more impermeable of the two materials and thus serve a useful purpose. Computations as made in the original report together with tests to determine the effect of shrinkage cracks are required to show the practical merit of such facings.

Mr. Mary questions the serviceability of permeability coefficients for correlating tests betwen various experimenters. Nevertheless by their use he has found that his specimens are apparently more permeable than those tested by the Bureau and has taken steps to discover the reason. His mixes, made at water-cement ratios which are high for American practice, appear to be about five times as permeable as similar specimens made by the Bureau at those ratios, but are appar-

ently only slightly above the results obtained by various other experimenters if the various approximations made to reduce their results to a common basis are reasonably correct. It is possible that Mr. Mary's method of placing the concrete in the molds was enough different from that used by the Bureau to produce substantial differences in results. In Table 3 are given permeability tests comparing the effect of vibration with the standard rodding method of placement. Vibration reduced the permeability to one-fifth that found for the standard method. Glanville reports a hundred per cent variation in permeability in certain of his mixes due to changes in the amount of rodding of specimens.

It is thought that conclusive evidence has been obtained concerning the disputed point as to whether or not permeability varies with the direction in which water percolates through concrete. Table 3 includes the results from a number of large companion specimens made of mixes varying from ¼ to 4½-in. maximum size aggregate, one test of each pair being made in the direction of pour and the other at right angles to it. The results indicate no variation in permeability coefficient with change in direction of flow.

Commercially it is important to know the effect of various periods of field curing with respect to permeability. While certain definite minimum requirements with respect to curing are required on account of such factors as strength, shrinkage, possible damage by freezing and the like, it has been suggested at times that additional curing would be advantageous from the viewpoint of percolation. On Fig. 15 are plotted the percolation curves for specimens made of a 1:2.72:2.78 mix with water-cement ratio of 0.60 in a series of tests made for the purpose of determining the effect of interrupted curing. The specimens were cured for varying periods, dried to constant weight at 120° F. in a ventilated heat chest and then tested for permeability. Three hours after the percolation test began, for initial curing periods of 1, 3, 7 and 14 days the discharge rates were in the relation of 25, 12, 5 and 1 respectively, showing the relative effect of the initial cure. These relative values are only approximate for, as already pointed out, steady flow had not been established. Comparing the specimens at an age when the period of initial cure plus the time under test is the same, thus approximating equivalent total cures, then for 28 days of total cure the discharge rates are in the relation of 3.5, 1.8, 0.8, 1.0. However a relative coefficient of only 0.25 at the end of 28 days was estimated for an undried companion specimen.

From these tests the following tentative conclusions may be drawn. If concretes of a given mix are completely dried once at 120°, within a

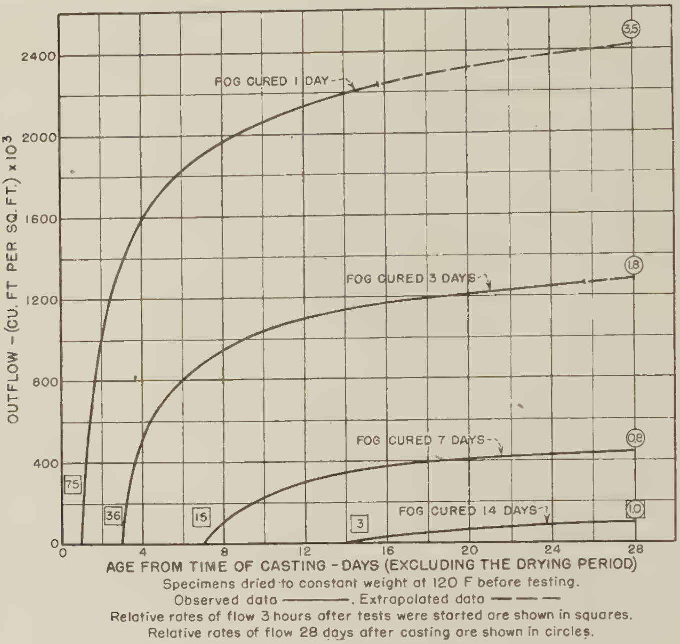

FIG. 15—EFFECT OF INTERRUPTED CURING

range of tests extending to 28 days, there will be but small differences in the final permeability regardless of the length of initial cure, providing the concrete has been cured at least 24 hours. However complete drying of the specimen at 120° F. is apparently accompanied by a change in the pore structure which permanently increases the permeability several hundred per cent within the time limits of the tests.

Since the publication of the original paper additional data have been obtained on the effect of leaching on the compressive strength of concrete. As very little information of this sort has been published, the results from all the tests are presented in Fig. 16. The tests were made on mixes varying from 1:6:11 to 1:10:18 with water-cement ratios varying from 1.2 to 2.0. The graph indicates that 16 to 20 per cent of the cement content of the concrete (measured by per cent removal of lime) may be removed without loss of strength, the 6 per cent lower strength shown being attributed to the saturated condition of the specimen. With one-third of the cement removed about 11 per cent of the initial strength of a saturated specimen has been lost, the rate of loss then being about 1 per cent per 1 per cent removal of

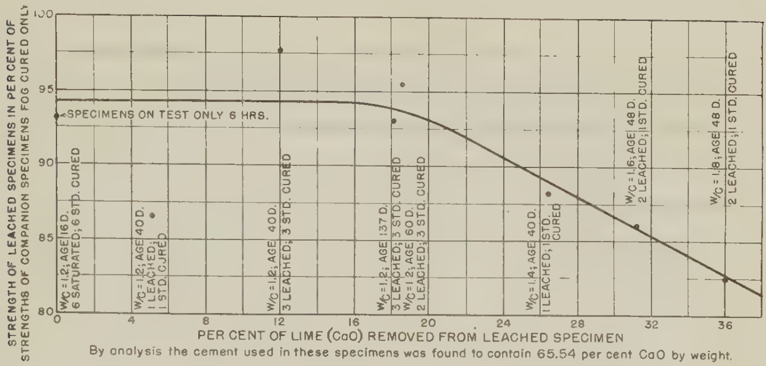

FIG. 16—EFFECT OF LEACHING ON COMPRESSIVE STRENGTH

cement. It is of interest that even with one-third of the original cement removed, the rate of flow through the specimen still is substantially constant.

The authors wish to apologize for an error made in abstracting data from the tests of Messrs. Norton and Pletta. On Fig. 9 these tests are referred to as "Wisconsin Gravel." The original Norton and Pletta test data are given on page 1125, JOURNAL of the American Concrete Institute, Vol. 27, and were tests made on 6-in. specimens at 40 pounds pressure and cured about 40 days. If it be estimated that the rate measured only 40 to 50 hours after the start of the test is three times that which would exist once the voids became saturated, and if from these values the coefficient with respect to paste is obtained by applying the same corrections for end effect and length of cure as in the Bureau's tests, the corrected coefficients will be about two and one half times the values plotted on Fig. 9.

Now that Boulder Dam has been completed and during the last six months has been exposed to some 300 ft. of water, it is deemed appropriate in closing to comment briefly on the behavior of the concrete, particularly in view of the opinions expressed by the authors in their original paper. Recent careful inspections of the galleries near the upstream face of the dam have shown that while there is indication of moisture along about 2 per cent of the construction and contraction joints exposed by the galleries and at half a dozen small isolated spots on the faces, at no point is there measurable leakage and the concrete as a mass is dry. It was for assurance of this result that the percolation tests were made.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Reinforced concrete slabs for air-raid protection

RUEHE, *Zement*, Vol. 24, No. 49, Dec. 5, 1935, p. 786-8. Reviewed by INGE LYSE.

This article discusses the necessity of constructing special vaults for the protection of the civil population and describes the type of construction used for a section of houses recently constructed in Breslau. The vaults are provided with two-way reinforced concrete slabs, designed not only for ordinary load conditions but also an additional load of 2500 kg. per sq. meter as prescribed by the police department.

Abrasion test for concrete

A. GUTTMANN, *Tonindustrie Zeitung*, Vol. 59, No. 77, p. 949-51, Sept. 23, 1935. Reviewed by A. E. BEITLICH.

A description is given of a simple device to be attached to the Boehme grinding table to make abrasion tests on concrete cubes. Slight differences between resistences against abrasion can be detected, such as are caused by the use of various brands of cement. The test is especially useful when aggregates are used whose resistence to abrasion is unknown.

Wind stresses in reinforced concrete arch bridges

A. A. EREMIN, *Proc. Am. Soc. C. E.*, Vol. 61, No. 10, Dec., 1935, p. 1453. Reviewed by H. J. GILKEY.

Wind stresses in arch bridges may be considerable, an extreme case being a 426-ft. arch bridge in Germany, in which wind stresses are 55 per cent of the total. While wind stresses frequently do not receive enough attention, there are some cases of heavy, unsightly and uneconomic design. The subject is treated mathematically for the arch rib without bracing and also for braced arch ribs. An illustrative example is given. Notation is covered in an appendix.

Reinforced concrete members under direct tension and bending

D. B. GUMENSKY, *Proc. Am. Soc. C. E.*, Vol. 61, No. 10, Dec., 1935, p. 1469. Reviewed by H. J. GILKEY.

Combined tension and bending has received less attention than has compression combined with bending. The problem assumes importance in the design of the Vierendeel truss and especially in the design of closed water conduits under pressure. The treatment is condensed, outlining the method and also indicating a type of diagram (in two different forms) that can be constructed to facilitate solutions.

The construction of silos by the Macdonald system of sliding forms

F. Streun, *LeGenie Civil*, Vol. CVII, No. 13, Sept. 28, 1935, p. 296-98. Reviewed by R. L. Bertin.

This article describes in detail this type of sliding forms and their application to the construction of 15 cell silos in Asnieres, France, which are 7 meters in diameter and 24.50 m. high, the walls of which are 10 cm. thick. This structure was erected at the average rate of 1.70 to 1.80 m. per day. The article is well illustrated.

This system of construction is exploited exclusively in France and her colonics by "Société de constructions en beton a coffrages glissants" of Paris.

Another article dealing with sliding forms was published in the May 28, 1932, issue of *LeGenie Civil*.

Methods for testing watertightness of concrete and mortars

Ragnar Schlyter, *Technisk Tidskrift*, Vol. 65, No. 52, Dec. 28, 1935. Reviewed by Inge Lyse.

This article describes the methods adopted by the Swedish Material Testing Laboratory for testing impermeability of mortar and concrete. The test specimens consist of cylindrical or octagonal prisms which are made with a hollow center section. The water pressure is applied in this section and the permeability is measured by determining the amount of water collected in a container. If the specimens show no leakage, the permeability is judged by inspection of how far the water has penetrated into the concrete. The water pressure varies with the age of the specimen and is applied for two days. The method provides for a simple and quick means of ascertaining the watertightness of the concrete used on any construction job.

Moving seven miles of pavement twelve feet sidewise

S. Johannesson, *Engineering News-Record*, Vol. 115, No. 23, Dec. 5, 1935, p. 767-71. Reviewed by N. M. Newmark.

A concrete pavement slab 14½ ft. wide, the western strip of a road 39½ ft. wide, is being shoved westward 12 ft. in 500 ft. sections to form a part of New Jersey State Highway No. 26, U. S. No. 1, south of New Brunswick, N. J. An additional 10 ft. slab will be constructed west of the moved slab, to form a 25 ft. roadway for south bound traffic. The remaining part of the old roadway will carry northbound traffic.

Movement of the slab is accomplished by ingenious use of fire hose inflated by compressed air. After the movement is completed the slab is raised to proper grade by mud-jacking. Complete details of the entire process are described.

Analysis of set concrete

Cement and Cement Manufacture, Vol. 8, No. 12, Dec., 1935, p. 303-6. Reviewed by J. C. Pearson.

This article describes in detail the Australian Standard method for the Analysis of Set Concrete. The procedure cannot be adequately reviewed in brief space, and this reference is given, along with two similar references to this same magazine, merely for the information of chemists and others who may be sufficiently interested to compare these methods with that of A. S. T. M. An English Method, supplied by two well known London firms, R. H. H. Stanger, and Woodcock and Mellersh, is presented in the April, 1935, issue, p. 108-113. Dr. C. R. Platzmann presents the method used in Germany under the title, Determination of the Proportions in Mortar and Concrete, in the Nov., 1935, issue, p. 257-260.

Dipping concrete roofing tiles

R. H. Baumgarten, *Concrete Building and Concrete Products*, Vol. 9, No. 1, Jan., 1936, p. 5-6. Reviewed by J. C. Pearson.

The dipping of concrete roofing tiles, after they have been removed from pallets, in solutions containing chemicals and pigments, appears to be common practice in

England. The purpose is to fix the color, to obtain uniform surfaces, and to color the edges of the tiles if this is not done in the manufacturing process. Ferrous sulphate solution with added pigment is commonly used for red, brown, yellow or black tiles. The author finds ferric chloride preferable to the sulphate, provided the slightly orange tint which this salt produces is not objectionable. Copper sulphate is used with green pigments. Details regarding the various solutions are given, also a description and photographs of the simple equipment required.

The protection of concrete in sea water

RICHARD GRUN, *Tonindustrie Zeitung*, Vol. 59, No. 96, 97 and 98, p. 1185-6, 1202-3, 1213-5, Nov. 28, Dec. 2, 5, 1935. Reviewed by A. E. BEITLICH.

The chemical methods, such as treatments with fluorides, carbon dioxide and oxalic acid are discussed and it is shown that the carbon dioxide treatment (exposure to air) is the simplest method which can be made more effective by artificial neutralization of the lime. Great density alone is no sufficient protection for concrete against aggressive sea water. Results with protective coatings which meet the specifications for such materials are compared with long-time tests of storage in the field and aggressive sulfate solutions. The various properties of coatings and their protective value are illustrated. Some consideration is given to the use of certain pigments in coatings and the application of the coatings upon various surfaces.

Cement for concrete roads

RICHARD GRUN, *Tonindustrie Zeitung*, Vol. 59, No. 72, p. 882-8, Sept. 5, 1935. Reviewed by A. E. BEITLICH.

A systematic investigation of cements as to their suitability for concrete road constructon revealed certain important facts which showed additional requirements beyond the standard specifications for such cements which are chiefly used in road work. Such requirements are a longer setting time, high flexural strength and low shrinkage. Cements which are too finely ground are not recommended. Other favorable properties are low heat of hydration, little shrinkage during cold or dry weather periods, good bond with aggregates and high elasticity. The author concludes in saying that a study of the properties involved and a careful selection of cements already tested in the field will yield a perfectly suitable cement for roads.

Ingenious relining procedure in a traffic-burdened tunnel

Engineering News-Record, Vol. 115, No. 23, Dec. 5, 1935, p. 774-9. Reviewed by N. M. NEWMARK.

The 6.21 mile Moffat railroad tunnel was originally lined with timber for more than half its length. Rapid deterioration due to frequent wetting and drying necessitated expensive maintenance; consequently it was decided to cover or replace the lining with reinforced concrete at an estimated cost of nearly $900,000.

The concrete lining was designed for a working strength at any point equal to the ultimate strength of the existing timber lining. Part of the timber is removed, the rest left in place and covered with a minimum thickness of 6 inches of concrete. Extreme caution must be observed in disturbing the original timbering. Train and smoke interference add to the difficulty of the work. In spite of these handicaps a steady progress of 40 ft. of complete tunnel per day has been maintained.

Wearing surface of concrete floors

E. SUENSON. Reprint of a Lecture given before Hospital Inspectors, Sept., 1934, Copenhagen. Reviewed by INGE LYSE.

Two of the important properties of wearing surfaces of floors are light weight and small thickness. Other problems for consideration are low cost, high durability,

pleasing appearance and such questions as sound and heat insulation, easy cleaning, water tightness, acid resistance, and fireproofing. These problems are discussed by Professor Suenson for different requirements. He continues with discussion of types of soft surfaces such as cork, rubber and linoleum, medium types such as asphalt, magnesia mortar, wood, wooden fiber blocks and asphalt blocks, and hard surfaces such as cement mortar, terrazzo, natural stone and artificial stone and clay blocks. Advantages as well as disadvantages are pointed out for the different surface materials with information of how to obtain the best results with each.

Design of concrete mixes for Mississippi River dams

RALPH P. JOHNSON, *Engineering News-Record*, Vol. 115, No. 22, Nov. 28, 1935, p. 743-6.
Reviewed by N. M. NEWMARK.

The remolding apparatus used by T. C. Powers in studies of workability (JOURNAL A. C. I., Vol. 27, 1932, p. 419) was used with a trial mix method in proportioning the mixes for some 430,000 cu. yd. of concrete in five locks and two dams of the St. Paul Engineer District. A description of the concrete requirements specified, the design of the mixes, and a discussion of the results obtained, are given in this article.

The mixes used at the various jobs were changed from time to time as the grading of the available aggregates changed. Uniform workability was maintained by the manner of designing the mix. The amount of cement used was, in general, the minimum specified, 4.5 bags per cu. yd. of concrete; and the water cement ratio averaged about 6 gals. per bag of cement. Compressive strength at 28 days averaged 3500 to 4000 p. s. i.

Lime modulus, lime saturation and lime standard of portland cements

HANS KUHL, *Tonindustrie Zeitung*, Vol. 59, No. 100, p. 1221-4, Dec. 12, 1935.
Reviewed by A. E. BEITLICH.

The author gives a critical discussion of a number of moduli, introduced into the cement chemistry by himself, Rordam, Forsén, Lea and Parker, and others, which express the ratio between the actual lime content of a portland cement and the highest theoretically possible lime content of the same cement. These moduli vary depending on which theories of cement constitution they are based and to which type of cement they are applied: portlands with high iron and low alumina content and such with low iron and high alumina content. He points out the slight differences between his formula and the one proposed by Lea and Parker. For future work, he suggests the use of the Lea and Parker formula which is: $CaO_{Stand} = 2.8\ SiO_2 + 1.18\ Al_2O_3 + 0.65\ Fe_2O_3$, with his definition which is: The lime standard is that amount of lime which can be combined by the hydraulic factors at the sintering temperature according to chemical equilibria.

Studies of equipment for compacting concrete in highways

OTTO GRAF, *Die Betonstrasse*, Vol. 10, No. 12, Dec., 1935, p. 245-50. Reviewed by INGE LYSE.

The enormous undertaking of Germany's governmental automobile highway system made it necessary to investigate the relative merits of different methods of placing concrete—among them vibration. Some of the questions raised were: (1) what types of equipment are available for highway concreting, (2) how many times should the vibration be applied, and (3) how thick a layer of concrete can be effectively vibrated into place? Professor Graf's report contains a description of the different vibrators used as well as a discussion of the methods employed and the test specimens produced. Four different vibrators were studied. Their weights varied

from 38 to 111 kg. and the speed of vibration from 1200 to 4000 per minute. The test specimens consisted of slabs and control cubes. The slabs were tested for modulus of rupture while the cubes were tested for compressive strength. Hand tamped slabs served as control. The results showed that the hand tamping gave essentially the same strength as did the vibration for gravel concrete. It was also found that one vibration period was as effective as three periods for gravel concrete, while for crushed stone concrete three periods of vibration increased the strength somewhat. The vibration method was also more effective than hand tamping for crushed stone concrete.

The effect of physical and chemical conditions upon the shrinkage of portland cements

HANS KUHL and DZUNG HSIEN LU, *Tonindustrie Zeitung*, Vol. 59, No. 70, 71, 74, 82 and 83, p. 843-5. 864-6, 913-5, 1016-8, 1028-9, Aug. 29, Sept. 2, 12, Oct. 10, 14, 1935. Reviewed by A. E. BEITLICH,

The shrinkage of 13 different cements of various compositions with and without admixtures of gypsum or calcium chloride were studied under several test conditions. Long storage reduces the tendency to shrink even in completely dry air. Larger amounts of mixing water reduce initial shrinkage and increase final shrinkage. The greatest shrinkage takes place in very dry air. Only very small differences in shrinkage are found as long as the moisture content of the air is between 0 and 40 per cent. Prolonged storage of the specimens in the moist closet retards the beginning of the shrinkage but does not decrease the final amount of shrinkage. In air-tight containers the specimens shrink very little. The silica content of the cements is of no noticeable effect upon the magnitude of shrinkage. A high alumina modulus increases the tendency to shrinkage. A Ferrari-cement shows the least shrinkage. The initial shrinkage of cements with low lime content is rather slow. A properly selected amount of gypsum can reduce the shrinkage; the correct amount of gypsum increases with the alumina modulus. Admixtures of calcium chloride increase shrinkage and expansion.

Crazing of cast stone and concrete

Concrete Building and Concrete Products, Vol. 9, No. 1, Jan., 1936, p. 10-12. Reviewed by J C. PEARSON.

An abstract from the 1934 Report of the Building Research Station states that the studies of shrinkage resulting from carbonation at different humidities have been continued. Since the absorption of CO_2 releases water inside the specimen, this may mask the effect of carbonation both by causing expansion and upsetting humidity control. In attempting to calculate volume changes from density determinations, it was found that carbonation of a hydrated cement produced nearly twice the calculated expansion, which could only be accounted for by the assumption that lime included in the cement gel is expelled during the carbonation process without an equivalent contraction of the remaining gel. In the testing of mortar specimens a net shrinkage is observed from carbonation, because a part of the calcium carbonate formed is accommodated in the pores of the specimen.

The report also calls attention to the fact that the effects of drying and carbonation shrinkage may be more apparent in crazing of cement products after long continued wet curing than if they take effect when the concrete is immature. This raises the question whether such products as cast stone, where appearance is of more importance than high strength, should not preferably be submitted to short curing rather than the traditional long periods of thorough curing.

Volume changes of cements

G. Mussgnug, *Zement*, Vol. 24, No. 45, Nov. 7, 1935, p. 717-21. Reviewed by Inge Lyse.

The primary purpose of an investigation of volume change of concrete was to find the effect of hydraulic and non-hydraulic admixtures on the concrete. The admixtures used were three types of slags, a highly active, a medium, and nearly inert slag, the silicious material trass, glass and quartz sand. The admixtures or blending materials amounted to about 30 per cent and were ground with the clinker to ordinary cement fineness (3 to 4 per cent retained on the 4900 mesh). The test specimens (4 by 4 by 20 in.) were made of concrete containing sand and gravel aggregates with a cement content of 400 kg. per cu. meter. Alternate wetting and drying and dry air storage were provided.

The results showed that the different ingredients in the cements produced relatively small effects on the shrinkage of the concrete, although a rich mix was used. Compressive and flexural tests showed that the cements containing the various types of slag gave essentially the same strength results as did the non-blended cement, while the other admixtures lowered the strengths.

A second series of tests was made to ascertain the effect of the observation method on the shrinkage. The results indicated that the observation of shrinkage of air stored specimens may not always give dependable information regarding the volume changes of the concrete.

Production of cement and iron in the rotary kiln

Adrian Margarit, *Cement and Cement Manufacture*, Vol. 8, No. 12, Dec., 1935, p. 281-3.
Reviewed by J. C. Pearson.

A new process for the simultaneous production of iron and portland cement clinker, based on the patent of a French chemist, M. Lucien Basset, is being operated near Barcelona. The iron-bearing raw material is burnt pyrites, a by-product in the manufacture of sulphuric acid and superphosphates, and very abundant in Spain. The pyrites are mixed with coal and the usual raw materials of cement, and this mixture, in the form of a fine powder slightly moistened, is introduced into a 9 x 140 ft. dry process Allis-Chalmers rotary kiln, only slightly modified for adaptation to the new process. The heat dries and calcines the calcareous material while CO reduces the ferric oxide and liberates the iron. The molten iron remains at the bottom covered by a layer of solid slag, the composition of which is that of good portland clinker if the mix has been properly made.

The clinker, carrying only 1 or 2 per cent of iron, reaches the lower end of the kiln after flowing over a ring dam which retains the liquid iron. At this point the kiln shell has a discharge hole permanently open, through which the iron discharges intermittently into a skip. The kiln is lined with clinker bricks which are self maintaining, with the aid of a spray cooling system at the zone of highest temperature. Both iron and cement are of high quality, according to test data and analyses given in the paper.

Simplification of tests for controlling the quality of concrete on small jobs

J. Bolomey, *Le Constructeur de Ciment Arme*, Nov., 1935. Reviewed by P. H. Bates.

The author suggests a procedure for controlling the quality of concrete on small jobs which he believes will be sufficiently simple and economical for general use. It is based upon the assumption that the compressive strength of the concrete can be

calculated with sufficient precision from the equation: compressive strength = $\left(\frac{C}{E} - 0.5\right) \times K$ in which $\frac{C}{E}$ is the ratio of cement and water by weight; K is a coefficient depending upon the nature and quality of the cement used, and the length of time of hardening. A table is presented giving the values of K for different cements aged for different periods. The values vary from 70 to 320 kg. per sq. cm. The amount of water for the mix is determined by the equation: Amount of water = $\frac{[1.000 - (c \times v)] \times D_s - (D - C)}{D_s - 1}$, in which c and v are the volumes per cubic meter of the cement and the voids; D is the "density" of the concrete when gaged; D_s is the absolute density of the cement; C is the weight of the cement per cubic meter of concrete. The density is determined by weighing some specially made vessels of known volume filled with concrete.

As a further measure of the strength of the concrete which may be determined readily at the job, the author suggests screening out all the coarse aggregate from a sample and using the mortar so obtained to make specimens 4 x 4 x 16 cm. These are broken on the job with a small portable hand-operated testing device. The author cites some data showing the relation between the strength in flexure of these small specimens with the compressive strength of the concrete.

A reinforced concrete hangar at the Seville airport (Spain)

ALPHONSE PENA BOEUF (Member of the Academy of Science of Spain), *LeGenie Civil*, Vol. CVII, No. 12, p. 269-70, Sept. 31, 1935. Reviewed by R. L. BERTIN.

This hangar, intended to house two air ships of the Zeppelin type, consists of a reinforced concrete vault 256 meters long, having a clear span of 126 m. and height at the center of 58 m.

It is closed at both ends by means of steel doors, each consisting of four sections shaped like an orange peel, two of which are fixed and two are rotated on counterweighted trucks running on circular tracks. The door sections do not touch or bear on the reinforced concrete vault.

The system of vault construction is similar to that used by Mr. Freyssinet for the hangars of Orly, France, described in the Sept. 22 and 29 and Oct. 6 issues of *LeGenie Civil.*

The vault is composed of 32 parabolic corrugations, 8 m. on centers, lengthwise of the vault and 4.20 m. deep. The external face of the corrugations are squared for a width of 2.50 m. and glazed for lighting to about two thirds of the height. In the central section this space is left open and is covered with ventilating skylights.

Although the walls of the corrugated ribs are very thin, the moment of inertia of the rib as a whole is large, thus obtaining a strong section with a minimum of material.

The ribs were computed to carry in addition to the dead load, the weight of the two Zeppelins hung from the ribs, and the most unfavorable combination of wind condition acting normal to the surface or inclined to it and acting internally and externally, assuming the doors to be opened.

For the construction of the ribs, a traveling steel scaffolding was used, the extrados of which was kept 1.50 m. from the intrados of the vault. Collapsible forms were used to form the corrugated ribs which when stripped were folded on the traveler, the whole being moved to the next rib and the collapsible forms re-erected.

The relation of certain anomalies of the time of set to the grinding of cement clinker

MARC ELBER, *La Revue des Materiaux de Construction et de Travaux Publics*, Nov., 1935.
Reviewed by P. H. BATES.

The author noted in the cement mill with which he was connected that a change in type of grinding equipment resulted in the cement acquiring what is referred to in the United States as "grab set." Determinations of the temperature of the cement from the new and old mills showed a much higher temperature in the case of the new mill. However, a small mill for laboratory grinding which was introduced in the laboratory to replace an older one, showed the cement prepared in the new laboratory mill also had "grab set," although there was no apparent rise in the temperature of the cement. The author therefore concluded that the development of "grab set" could not be attributed to the dehydration of the gypsum alone. Some further work resulted in his noting that in the laboratory mill the fineness of grinding did not increase with the time of grinding beyond about two hours. At this period the residue on a 4900 mesh sieve was 4.2 per cent whereas after 48 hours grinding the residue was 22.4 per cent. It was quite evident that increased grinding resulted in increased agglomeration of the cement in the mill. The increased agglomeration was accompanied by increased evidence of "grab set." Analytical work showed that the cement retained on the sieves contained practically all the added gypsum. The author concluded that in the mills the gypsum at first possibly dehydrated, then absorbed moisture and gathered about it fine particles of cement, and in so doing increased the amount of residue on the sieve.

With this line of reasoning the author then proceeded to regrind the coarse particles of cement but found it necessary to modify his theory in view of the fact that this cement produced by grinding the coarse particles high in gypsum still had "grab set." The explanation then offered is that in the grinding in the new mill there is a change of structure of the gypsum molecule, which he infers is somewhat analogous to cold working of metals in which a complete rearrangement of the molecular structure is brought about.

Laboratories of Building and Public Works (of France)

Special Number of *L'Entreprise Francaise*, Nov., 1935; (9, Avenue Victoria, Paris IVe 50 francs, post paid).
Reviewed by P. H. BATES.

A full description of the Building and Public Works Laboratories, 12, Rue Brancion, Paris, dedicated June 21, 1935, by M. Herriot, President of the French Association for the Development of Technical Education, and M. Luc, General Director of Technical Education. This special number of *L'Enterprise Francaise* with 284 pages and 32 plates is intended to show to those who are directly or indirectly interested in the physical and chemical qualities of building materials, the modern means of investigation which these new and very adequately equipped laboratories place at the disposal of the French people.

It contains addresses by Messrs. Lassalle, Caquot, Maigrot, Luc and Herriot, all outstanding figures in the educational and constructional groups of France. Construction features of the various individual laboratories are described with an exposition of the work done in these laboratories by those instrumental in their design and equipment.

Considerable space is directed to informing the public on how it may use the facilities of the new laboratories. This is indicated through "Utilization tables," which show what tests are most adapted to the research or control desired. There

are subdivisions corresponding to a class of materials and successive divisions in each class which bring the general case to a specific one. A catalogue of tests is also presented, so subdivided as to correspond to the class of materials in the "Utilization tables." Application forms for the necessary tests are also shown. For facility of payment, stamp books are sold by the laboratory and payments are made by sticking the necessary number of stamps on the back of the order sheet.

Members of this Institute will be particularly interested in the 5,000,000 (?) pounds testing machine which forms one of the features of the laboratory. Largely designed upon suggestions by Freyssinet, concrete enters into its construction to a major degree. Essentially, the machine is a large reinforced oval concrete cylinder lying on its side. The interior of the cylinder provides (14 m. long, 4.3 m. high and 3.6 m. wide) not only for loading jacks, the lower platen of the machine, the gages, and all equipment for operation, but space also for the operators. The loading jacks are placed in the concrete ceiling and the lower platen on the concrete floor of the cylinder; hence the flattened side walls take the entire reaction of any imposed load on a specimen. That the concrete is able to withstand such stresses is due to the application of Freyssinet's suggestion, namely, that of compressing the newly placed concrete and holding it under compression at all times through the procedure of placing the reinforcing (high elastic alloy steel) in tension.

Vibrated concrete—consideration of the different methods

ETIENNE TREVES, *LeGenie Civil*, Vol. CVII, No. 16, Oct. 19, 1935, p. 366-71.
Reviewed by R. L. BERTIN

The author presents a historical review of the development of vibrating concrete which he claims originated in France.

In 1919, the Limousin Co. (Freyssinet System), applied for a patent in France which contained in principle several features of concrete vibrating as practiced today, particularly external vibration by means of vibrating the form work and vibration in the mass by means of hand tools consisting of plates or bars carrying a vibrating element. In the text of the patents, the necessity of high frequency was stressed.

In 1928, the Monod Co. vibrated the concrete of the Lafayette Bridge in Paris (*LeGenie Civil*, Dec. 1, 1928, page 517) using compressed air vibrators. At the same time, vibrators were used on curb forms in Paris.

The use of vibration from then on increased rapidly. Internal vibration or pervibration entered the field in the experimental stage, and surface vibration, easier to apply, was demonstrating itself superior to hand compacting in the construction of concrete roads and large masses. During the last five years, vibration has been adopted generally throughout the world.

Much technical literature exists on the subject but most of it is narrowed down to specific apparatus or particular applications, or written by authors with preconceived ideas so that the involved fundamental principles are not easily drawn therefrom.

External Vibration. The application requires experience. Depending upon the results desired, be it impermeability, early stripping, maximum strength, fast pouring, different methods must be used.

External vibrators are classified under three headings:

(1) Hand tools
(2) Vibrating tables
(3) Various types of pneumatic or electric vibrators

Hand tools, such as pneumatic hammers applied manually to the forms, or vibrators on the ends of long handles also applied to the forms, are helpful but are not,

strictly speaking vibration, the number of cycles being far too low (1200 to 1600 per minute instead of 4000 to 10000). These tools are unsatisfactory because they depend on the judgment of the workmen.

The vibrating tables have a limited application, namely precast units.

The pneumatic or electric vibrators equipped with various means of attaching them to the forms are suitable for all conditions so long as the thickness of the concrete is not excessive. In the latest models of pneumatic vibrators, the oscillations vary from 5400 to 21000 per minute, the total weight has been reduced and the efficiency increased. The method of attaching to the forms is also greatly simplified.

Electric vibrators with unbalanced rotors acting tangentially instead of normally to the forms are of necessity heavier and more bulky.

The efficiency of external vibrators are based on two criteria:

(1) The time required to obtain a given result
(2) The radius of action of the apparatus.

These criteria are measured relatively by means of a special apparatus consistin of a long box (sizes not given), at the end of which the vibrators are attached externally. Along the length of the box hollow spheres of thin metal with rods attached thereto are equally spaced, the sphere resting on the bottom of the box and the rods set vertically, passing through holes in a longitudinal plate at the top of the box, acting as guides for the rods. The box is then filled with concrete and the vibrator set in motion. The sphere begins to rise and the time required for each sphere to come to the surface is recorded. The concrete block is subsequently tested for strength.

Such tests are being conducted at the Ecole Centrale des Arts & Manufactures by Mr. Galibourg and will be the subject of a paper to be published later. Certain conclusions are given, as follows:

Concrete containing at least 130 liters of water per cubic meter, pneumatic or electric vibrators give analogous results.

With concrete containing from 100 to 110 liters of water, the electric vibrator fails to cause the spheres to rise, whereas the pneumatic one does. Well illustrated applications of vibrators are given. Interesting decorative effects were obtained by vibrating concrete, stripping early, and sand blasting the surface.

Surface Vibration. This is the only possible method for large surfaces of thin section, such as roads, and is the most economical and rapid for large masses used singly or in combination with other means.

In large masses where impermeability is of prime importance and strength secondary, external vibration of the forms is sufficient.

A form of surface vibrators with open platform prevents tearing the concrete surface already vibrated which occurs with solid platforms.

Vibrating rollers are also described, in some of which the vibrating element revolves with the roller and others in which the element is always vertical and in contact with the lower generatrix of the roller.

Internal Vibration is divided into two groups of apparatus:

(1) Those held by hand and moved about in the concrete
(2) The floating apparatus or pervibrator

The first are in the public domain, having been clearly anticipated in the Limousin patents of 1919. A number of these exist especially in the United States, some of which are pneumatic, others electric. The author's objection to these is that they involve the personal coefficient of the workman and their radius of effectiveness is

small. He recognizes their value, however, in cases of large beams too wide for external vibration or too heavily reinforced for the application of surface vibrators.

The pervibrators, devised by M. Deniau in 1927 consist of shells of sheet iron containing a vibrator. Immersed in the concrete, they vibrate it and rise automatically. Their use is limited to sections without cross reinforcement.

The author concludes by calling attention to the necessity of a general study of the subject in the laboratories as well as in the field, with a view of creating a better understanding of the proper application of vibrating tools to concrete under the various field conditions.

At the Forks

BY P. H. BATES
RETIRING PRESIDENT, AMERICAN CONCRETE INSTITUTE,
AT 32ND ANNUAL CONVENTION

Locally, nationally, and internationally human thoughts and purposes are now at a "forks in the road." In practically all of our endeavors, it is acknowledged that things have not been what they should have been. In some matters it might appear that a new course has been set and the road to the right is to be followed—but there is always a powerful minority which contends that the *right* is in reality the *left*. In but few cases has there been a joining of the extreme viewpoints and the surveying of a new road—possibly somewhat intermediate between the extremes of the left and the right.

While you are doubtlessly thinking that these remarks concern only economics and politics, they are put forward as applying also to industrial and to association activities as well. Mentally and physically the world as a whole in most of its efforts has been for some time in pretty much of a fog of uncertainty, not knowing just where, how or when it should direct its efforts—and we must include in this world of hesitants the concrete industry and the American Concrete Institute.

Not many of those present can recall the status or course of the concrete industry three or four decades ago. Actually it did not have much of a well defined course at the beginning of the century. The industry was waddling around in its swaddling clothes and but a mite of an impediment in its path might send it off at a tangent or back over the way it had just came. The uncertainty of the interested few at that time was a matter of much moment—the uncertainty today of the interested many is a matter of still greater moment.

Our recollections of the early concrete in this country tell us that it was made of whatever cement was at hand ("whatever cement" does not mean brand of portland cement—but type of hydraulic cement—portland, natural, slag, the blends of these and the results of the poorly controlled efforts at making any one of these varieties), whatever fine aggregate was at hand, and whatever coarse material might be available. The amounts of water—as little as possible—and

the character of the placing—through ramming—were outstanding high spots in the technique of concreting, but these excellent procedures had been determined with no apparent reasoning and were hence eagerly dropped for procedures no more reasonably derived but more easily used and almost thoroughly wrong.

The early concrete industry was, let us say, highly speculative in nature. Those few who were designing our early reinforced concrete structures were basing their designs upon decidedly speculative premises. The deportment and the service of the material was based upon a faith in the future rather than in any extended evidence of a past. The rapid expansion in the use of reinforced concrete brought about many changes in the early industry—more than did the introduction of the concrete highways at a later period. It came at a time when concreting was relatively so young and little used that these somewhat daring applications attracted much attention. Young engineers were impelled either to study the material in the laboratory or apply it in the field. At the same time those interested in construction directed their attention to it. There resulted a rapid mechanical development in handling and placing. Indeed, it is now realized that this development was too rapid and much of it based upon erroneous premises.

In time there gradually evolved from the somewhat poorly defined conditions of how structures and concrete should be designed, how the latter should be handled, placed and treated during its hardening, certain generally accepted procedures—including the recognition of the need of studied design, close inspection and frequent testing. Naturally there developed an interest and a literature which attracted more new thoughtful ones into the field and still more thought on the part of those already within it. But it seems that this new thought on the part of an ever increasing number only served to emphasize that what had been thought to be certain was still a matter of conjecture and that what had been thought to be good practice was, in reality, open to some decided questioning. This has extended to date and even to that most essential ingredient of concrete—cement—and at no time has there been so much uncertainty as to what should be the nature of the cement for any purpose than now.

Those who have been thinking faster than I have been speaking or those who have stopped questioningly at some statement and thus, having lost the drift of the discussion, have been led to soliloquize on their past, are doubtlessly asking what has this swirling and whirling

of the development of concrete to do with the present position of the American Concrete Institute.

Until the beginning of the eventful "30's" of this century, the Institute had a history rather strikingly similar to that of the industry in which it is so much interested. But when that industry, like practically all others, suddenly "brought up short" and went "into a huddle" the Institute did likewise. Similarly it found itself in rather a precarious state. But through careful thought and management it has come through the few lean years well seasoned for its properly directed future activities. The question is, what should be the direction of the future activities.

The Institute is a group of individuals. To continue to live it must interest individuals—those interested in concrete. If we look upon concrete as a finished commodity, then the groups to be interested are the cement producers, the producers of aggregates, the producer who makes concrete, either for a concrete structural unit costing a few cents or a few million dollars, those who make the machinery for mixing and placing concrete, the designers of the structures, those who build the structures, those who carry out investigations in the laboratory, those who do the inspecting, the testing, etc. When we analyze the membership of the Institute, we find that the interest of some of these is almost at the vanishing point. Even the cement producers have, to a surprising degree, cooled in their support of the Institute. The natural query then is—has the Institute failed to furnish its variety of members with the variety of interest that will hold their attention. The answer is that it possibly has. How then can it again secure their attention?

The privilege of paying the annual dues will not hold members. They want an adequate return for their dues. To give them their returns the Institute must get the returns for them and the best way to get these returns for them is to get these returns from them—illustrating that it is more than "the music goes around and around." Tersely the Institute must put its members (and even non-members in the hope that they will become members) to work to develop the returns which get and hold members. How can it do this—through its meetings, its publications and its committee activities.

Do our meetings have the appeal to the variety of interests that the Institute's field covers? I think you will agree that they do not. I can find but little to attract me to a meeting given over to a discussion of the relative value of this kind of a mixer, that kind of a vibrator and the merits of the varieties of methods for placing concrete. But I do

concede that there are surely several thousand persons in the United States who are much interested in such matters. Do these several thousand have much interest in the principles of the design of flat slabs—surely not enough to assure getting a baker's dozen of them to attend a meeting where such a subject might be discussed by the best engineers of the country. We must broaden our meetings until we have not one but several papers for each of the variety of interests—and there are about a half dozen of these—that make up the Institute. Can that be done in a two or three day convention? Surely—by holding simultaneous meetings. Do we have sufficient funds to permit holding meetings of this type? Possibly not in sight. But let us gamble not once but several times, hold such meetings after intensive publicity and see if we cannot attract enough new members into the Institute to come out ahead in a short time. Definitely I would suggest the grouping of the Institute into divisions—such as a Division of Design, a Division of Materials, of Equipment, of Research, of Construction, etc., under the leadership of competent chairmen or vice-presidents of the Institute.

Just as an analysis of our meetings shows a lack in the needed broad appeal, so will an analysis of our publications show a similar want. But if we acknowledge this in respect to the JOURNAL, how about the other publications of the Institute? An effort has been made to "get going" the publication of standards and instructive manuals. But the "going" has hardly as yet, produced the results. The individuals or committees who should have been "going" have not been doing so. Why?—largely because the Institute has not produced enough consistent worth while results to engender in its committees the *want* to produce something worth while and to engender in individuals the *want* to be members of worth-while committees and to insist that the latter do worth-while work. We should take a few lessons from the American Society for Testing Materials but while doing so, incorporate a few most desireable changes. Not only should the fields to be covered by specifications and manuals be defined and committees to cover them be appointed but annual reports *must* be demanded, and if not forthcoming the responsible committee should be discharged immediately. Each committee should be given a definite time within which to present its first results for promulgation and if these are not forthcoming, discharge the committee and reorganize it. Lastly, fix a period—three or five years—after promulgation at which all specifications become automatically obsolete unless the committee can produce data showing that it should be continued for not more than a one year period. It is acknowledged that such committee activities

will take a great deal of time at the annual meetings but what of it? Let us even go to the extent of having semi-annual meetings, one of which might well be given over to committee activities alone.

I believe that the revival of interest in the Institute and by the Institute can best be made through committee activities. My thought as we hesitate "at the forks of the road" is to follow that road which will take us into a field of vastly more extensive and intensive committee work than has ever been attempted before by the Institute, under far more rigidly defined procedures than have been used and under policing by an executive committee of the Institute that would —may we say—force results. Acknowledging that this will be costly, let us plan these activities so that individuals and groups cannot fail to afford the cost to the Institute and to themselves.

Building Regulations for Reinforced Concrete
(A. C. I. 501-36T)

Proposed by Committee 501, Standard Building Code, the new "code" is here presented as revised and tentatively adopted, 32nd Annual Convention, American Concrete Institute, Feb. 25, 1936.*

CHAPTER 1—GENERAL

101—Scope

(*a*) These regulations cover the use of reinforced concrete and plain concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of these materials. In all matters pertaining to design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102—Permits and Drawings

(*a*) Drawings and typical details of all reinforced concrete construction showing the sizes and position of all structural members, metal reinforcement, design strength of concrete, and the live load used in the design shall be filed with the building department as a permanent record before a permit to construct such work shall be issued. Calculations pertaining to the design shall be filed with the drawings when required by the Commissioner of Buildings.

103—Special Systems of Reinforced Concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with these regulations or not covered by them, shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

104—Definitions and Notations

(*a*) *Definitions*—The following terms are defined for use in this code:

*In the preparation of its report (briefly outlined Journal Am. Concrete Inst., Nov.-Dec., 1935 and distributed in full in a separate pamphlet) the Institute's Committee 501, worked with the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute.

Aggregate—Inert material which is mixed with portland cement and water to produce concrete.

Column—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip—A portion of a flat slab panel one-half panel in width occupying the two quarter-panel areas outside of the middle strip, and extending through the panel in the direction in which bending moments are being considered.

Combination Column—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column—A column in which a steel or cast-iron section is completely encased in concrete containing reinforcement of spiral reinforcement and longitudinal bars.

Concrete—A mixture of portland cement, fine aggregate, coarse aggregate and water.

Deformed Bar—Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling so as to firmly engage the surrounding concrete. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 may be rated as a deformed bar.

Diagonal Band—A group of bars covering a width approximately 0.4 the average span, symmetrical with respect to the diagonal running from corner to corner of the panel of a flat slab.

Direct Band—A group of bars, covering a width approximately 0.4 l_1, symmetrical with respect to the center lines of the supporting columns of a flat slab.

Dropped Panel—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of a slab or beam.

Effective Area of Reinforcement—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Flat Slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting columns.

Middle Strip—A portion of a flat slab panel one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which bending moments are being considered.

Paneled Ceiling—The ceiling of a flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length—The distance along a panel side from center to center of columns of a flat slab.

Pedestal—An upright compression member whose height does not exceed three times its least lateral dimension.

Plain Concrete—Concrete without metal reinforcement; or reinforced only for shrinkage or temperature changes.

Ratio of Reinforcement—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete at that section.

Reinforced Concrete—Concrete in which metal other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

Surface Water—The water carried by the aggregate except that held by absorption within the aggregate particles themselves.

(*b*) *Notations*—The symbols and notations used in these regulations are defined as follows:

a = Width of face of column or pedestal.

α = Angle between inclined web bars and axis of beam.

A = Total area of top of pedestal, pier, or footing at the column base; the span length between opposite supports in one direction (floors with supports on four sides).

A' = Loaded area of pedestal, pier, or footing at the column base.

A_c = Area of core of a spirally-reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally-reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in tension in beams or in compression in columns; the effective cross-sectional area of reintorcement which crosses any of the principal design sections of a flat slab.

A_v = Total area of web reinforcement in tension within a distance of s (measured perpendicular to the direction of the web reinforcement bar), or the total area of all bars bent up in any one plane.

b = Width of rectangular beam or width of flange of T-beam.

b' = Thickness of web in beams of I or T sections.

b_1 = Dimension of the dropped panel of a flat slab in the direction parallel to l_1.

B = Span at right angles to span A (floors with supports on four sides).

c = Diameter, in feet, of column capital of a flat slab at the underside of the slab, or dropped panel. No portion of the column capital shall be considered for structural purposes which lies outside of the largest 90° cone that can be included within the outlines of the column capital; distance from gravity axis to extreme fiber in compression (in a column).

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the least lateral dimension of a concrete column; the diameter of a round bar or side of a square bar.

D = Deflection of a floor member under test load.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$\left.\begin{matrix} e_A \\ e_B \end{matrix}\right\}$ = Plate action factors for spans A and B respectively. = (Floors with supports on four sides.)

E_c = Modulus of elasticity of concrete in compression.

E_s = Modulus of elasticity of steel in tension or compression (30,000,000 lb. per sq. in.)

f_c = Compressive unit stress in extreme fiber of concrete in flexure or axial compression in concrete in columns.

f'_c = Ultimate compressive strength of concrete *usually* at age of 28 days. (Sec. 301 (*a*)).

f_r = Permissible unit stress in the metal core of a composite column.

f'_r = Permissible unit stress on unencased steel columns and pipe columns.

f_s = Tensile unit stress in longitudinal reinforcement; nominal working stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

f_v = Tensile unit stress in web reinforcement.

F_A = Ratio of the distance between assumed inflection points of the span A to span A in an isolated strip extending the entire width of the structure when a uniformly distributed load is applied to span A only (floors with supports on four sides).

F_B = Ratio as defined above, but applying to Span B (floors with supports on four sides).

$\left.\begin{matrix} F_A A \\ F_B B \end{matrix}\right\}$ = The distances, assumed for purposes of load distribution, between inflection points in spans A and B respectively (floors with supports on four sides).

h = Unsupported length of column.

I = Moment of inertia of a section about the neutral axis for bending.

j = Ratio of distance between centroid of compression and centroid of tension to the depth (d).

K = Least radius of gyration of a metal pipe section (in pipe columns); the stiffness factor, that is, the moment of inertia divided by the span (floors with supports on four sides).

K_A = Stiffness factor $\left(\frac{I}{A}\right)$ for span A of panel AB (floors with supports on four sides).

K_B = Stiffness factor $\left(\frac{I}{B}\right)$ for span B of panel AB (floors with supports on four sides).

$\left.\begin{matrix} K_{AR} \\ K_{AL} \end{matrix}\right\}$ = Stiffness factors for spans to right and left respectively of span A (floors with supports on four sides).

$\left.\begin{matrix} K_{BR} \\ K_{BL} \end{matrix}\right\}$ = Stiffness factors for spans to right and left respectively of span B (floors with supports on four sides).

l = Span length of beam or slab; span length of flat slab (usually expressed in feet) center to center of columns in the direction in which moments are considered (see Sec. 1003).

l_1 = Span length of flat slab panel center to center of columns, perpendicular to the rectangular direction in which moments are considered.

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment (see Sec. 701).

L = Span of member under load test. (See Sec. 202.)

M = Bending moment or moment of resistance in general.

M_o = Sum of positive and negative bending moments at the principal design sections of a panel of a flat slab.

n = Ratio of modulus of elasticity of steel to that of concrete = E_s/E_c.

N = The sum of the lengths of those edges of panel AB supporting continuous adjacent spans (floors with supports on four sides).

Σ_o = Sum of perimeters of bars in one set.

p = Ratio of effective area of tensile reinforcement to effective area of concrete in beams.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g (see Sec. 1103).

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

P = Total allowable axial load on a column whose length does not exceed 10

times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

r_a = Permissible unit working stress in concrete over the loaded area of a pedestal, pier, or footing.

r_A, r_B = Load distribution factors, that is, the proportion of total load ($w'AB$) carried in the directions A and B respectively (floors supported on four sides).

R = Least radius of gyration of a section; ratio of gross area to core area of a spirally-reinforced concrete column, A_g/A_c.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

t = Thickness of the flange of T-beams; the total thickness or depth of a member under load test.

t_1 = Thickness of flat slab without dropped panels; or the thickness of flat slabs, including dropped panels where such are used.

t_2 = Thickness of flat slab with dropped panels at points outside the dropped panel.

t_3 = Total thickness of slab (floors supported on four sides).

u = Bond stress per unit of surface area of bar.

v = Shearing unit stress.

v_c = Unit shearing stress permitted on the concrete of the web.

V = Total shear.

V' = Excess of the total shear over that permitted on the concrete.

w = Uniformly distributed load per unit of length of beam or slab.

w' = Uniformly distributed dead and live load per unit of area of a floor or roof.

W = Total dead and live load uniformly distributed over a single panel area.

CHAPTER 2—MATERIALS AND TESTS

201—Tests

(*a*) The Commissioner of Buildings, or his authorized representative shall have the right to order the test of any material entering into concrete or reinforced concrete when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(*b*) Tests of materials and of concrete shall be made in accordance with the requirements of the American Society for Testing Materials as noted elsewhere in this chapter. The complete records of such tests shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work, and shall be preserved by the engineer or architect for two years after the completion of the structure.

202—Load Tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure

shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given by the following:

$$D = \frac{.001\ L^2}{12t} \text{ in which} \qquad (1)$$

L is the span, t is the total depth of the slab or beam and D is the maximum deflection—all expressed in the same units.

If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the slabs or beams show a recovery of at least seventy-five per cent of the observed deflection.

203—Supervision

(*a*) All concrete work shall be supervised by the architect or engineer responsible for its design, or by a competent representative responsible to the architect or engineer. A record shall be kept of such supervision, which record shall cover the quality and quantity of concrete materials, the mixing and placing of the concrete, and the placing of the reinforcing steel. A complete record shall also be kept of the progress of the work and of the temperatures, when these fall below 40 degrees F., and of the protection given to the concrete while curing. This record shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work and shall be preserved by the architect or engineer for two years after the completion of the work.

204—Portland Cement

(*a*) Portland cement shall conform to the "Standard Specifications for Portland Cement" (A. S. T. M. Serial Designation: C9-30) or the "Tentative Specifications for High-Early Strength Portland Cement" (A. S. T. M. Serial Designation: C74-30T).

205—Concrete Aggregates

(*a*) Concrete aggregates shall conform to the "Tentative Specifications for Concrete Aggregates" (A. S. T. M Serial Designation: C33-31T). In localities where aggregates conforming to these specifications are not obtainable, aggregates that have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness and wearing qualities may be used under Sec. 302(a) Method 2, where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate shall be not larger than one-fifth of the uarrowest dimension between forms of the member for which the concrete is to be nsed nor larger than three-fourths of the minimum clear spacing between reinforcing bars.

206—Water

(*a*) Water used in mixing concrete shall be clean, and free from deleterious amounts of acids, alkalis, or organic materials.

207—Metal Reinforcement

(*a*) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" (A. S. T. M. Serial Designation: A15-35), or for "Rail-Steel Concrete Reinforcement Bars" (A. S. T. M. Serial Designation: A16-35).

(*b*) Welded wire fabric or cold-drawn wire for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (A. S. T. M. Serial Designation: A82-34).

(*c*) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Buildings" (A. S. T. M. Serial Designation: A9-34).

(*d*) Cast-iron sections for composite columns shall conform to the "Standard Specifications for Cast-Iron Pipe and Special Castings" (A. S. T. M. Serial Designation: A44-04).

208—Storage of Materials

(*a*) Cement and aggregates shall be stored at the work in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used for concrete.

CHAPTER 3—CONCRETE QUALITY AND WORKING STRESSES

301—Concrete Quality

(*a*) For the design of reinforced concrete structures, the value of f'_c used for determining the working stresses as stipulated in Section 305 shall be based on the specified minimum ultimate 28-day compressive strength of the concrete, or on the specified minimum ultimate compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) All concrete exposed to the weather shall have a minimum ultimate 28-day compressive strength of not less than 3000 lb. per sq. in.*

302—Determination of Strength-Quality of Materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(a). Method 2 shall be employed when artificial aggregates or admixtures are used.

TABLE 302 (*a*)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Content U. S. Gallons Per 94-lb. Sack of Cement	Assumed Compressive Strength at 28 Days p. s. i.
7½	2000
6¾	2500
6	3000
5	3750

NOTE: In interpreting this table, surface water contained in the aggregate must be included as part of the mixing water in computing the water-content.

Method 2—Controlled Concrete:

Proportions of the materials and water-content, other than those shown in Table 302(a) may be used provided that the strength-quality of the materials proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A. S. T. M. Serial Designation: C39-33). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

*In climates where weather is not severe this section should be omitted.

The curve shall be established by at least four points, each point representing average values from at least four test specimens. The water-content used in the concrete for the structure as determined from the curve, shall correspond to a strength which is twenty per cent greater than that called for on the plans, for concrete of a compressive strength less than 2500 lb. per sq. in., and fifteen per cent greater for concrete of a compressive strength of 2500 lb. per sq. in. or more. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

303—Tests on Concrete

(*a*) The Commissioner of Buildings shall at his discretion require a reasonable number of compression tests to be made during the progress of the work. Such tests shall be made in accordance with the "Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field" (A. S. T. M. Serial Designation: C31-33).

(*b*) Laboratory control tests and field control tests, or laboratory control tests only may be required. Not less than the number of specimens shown in the following table shall be made for each type of test, and not less than three specimens for each type of test for each strength of concrete for any one day's operation.

Total Cubic Yards of Concrete Placed	Minimum Number of Test Specimens
0— 100	One for each 50 cu. yd.
100—1000	One for each 125 cu. yd.
1000—2000	One for each 175 cu. yd.
2000—and over	One for each 250 cu. yd.

(*c*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7 and 28-day strengths of the concrete is established by test for the materials and proportions used.

(*d*) In all cases where the average strength of the Laboratory control cylinders shown by these tests for any portion of the structure falls below the minimum ultimate compressive strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mix or in the water-content for the remaining portion of the structure. In cases where the average strength of the cylinders cured on the job falls below the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength and may require load tests to be made on the portions of the building so affected as specified in Section 202.

304—Concrete Proportions and Consistency

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than 30 per cent nor greater than 50 per cent of the total unless otherwise required by the Commissioner of Buildings, except that these proportions do not necessarily apply to light-weight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work. Measurement of materials for Ready Mixed Concrete shall conform to the "Tentative Specifications for Ready Mixed Concrete" (A. S. T. M. Serial Designation: C94-35).*

*Wherever practicable such measurement shall be by weight rather than by volume.

305—Allowable Unit Stresses in Concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the following values, where f'_c equals the minimum ultimate compressive strength at 28 days.

Description		Allowable Unit Stresses: For Any Strength of Concrete as Fixed by Test in Accordance with Sec. 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Sec. 302: f'_c = 2000 p. s. i. n = 15	f'_c = 2500 p. s. i. n = 12	f'_c = 3000 p. s. i. n = 10	f'_c = 3750 p. s. i. n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.40f'_c$	800	1000	1200	1500
Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement, but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
Flat slabs at distance d from edge of column capital or dropped panel	v_c	$0.03f'_c$	60	75	90	113
Footings where longitudinal bars have no special anchorage	v_c	$0.02f'_c$	40	50	60	75
Footings where longitudinal bars have special anchorage	v_c	$0.03f'_c$	60	75	90	113
Bond: u						
In beams and slabs and one-way footings:						
Plain bars or structural shapes	u	$0.04f'_c$	80	100	120	150
Deformed bars	u	$0.05f'_c$	100	125	150	188
In two-way footings:						
Plain bars or structural shapes	u	$0.03f'_c$	60	75	90	113
Deformed bars	u	$0.0375f'_c$	75	94	112	141
(Where special anchorage is provided (see Sec. 903a), one and one-half times these values in bond may be used)						
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area*	f_c	$0.375f'_c$	750	938	1125	1405
Pedestals (see Sec. 1205)	r_a					
Axial Compression: f_c						
In columns with lateral ties	f_c	$0.154f'_c$	308	385	462	578
In columns with continuous spirals	f_c	$0.22f'_c$	440	550	660	825

*The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

306—Allowable Unit Stresses in Reinforcement

(*a*) The following unit stresses in reinforcing steel shall not be exceeded:

Tension:

Intermediate grade billet steel . . . . (f_s) = 20,000 p. s. i.

Rail steel bars . . . . (f_s) = 20,000 p. s. i.

Web reinforcement . . . . (f_s) = 20,000 p. s. i.

Structural steel shapes . . . . (f_s) = 18,000 p. s. i.
Wire mesh or other steel reinforcement, not exceeding ¼ inch in diameter (used in one-way slabs), fifty per cent of the minimum yield point as established by the A. S. T. M. Standards for the particular grade of steel used; but not to exceed . . . . (f_s) = 30,000 p. s. i.

Compression
Structural Steel section in composite columns . . . . 16,000 p. s. i.
Cast-Iron section in composite columns . . . . 10,000 p. s. i.

Note: If special conditions require the use of Billet-Steel Concrete Reinforcement Bars of structural or hard grades, the allowable unit stress shall not exceed 18,000 p. s. i. for structural grade nor 20,000 p. s. i. for hard grade bars.

CHAPTER 4—MIXING AND PLACING CONCRETE

401—Preparation of Equipment and Place of Deposit

(*a*) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the places to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and masonry filler units that will be in contact with concrete shall be well drenched (except in freezing weather), and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(*b*) Water shall be removed from place of deposit before concrete is placed unless otherwise permitted by the Commissioner of Buildings.

402—Mixing of Concrete

(*a*) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(*b*) For job mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer. A longer mixing period may be required for mixers larger than one cubic yard capacity.

(*c*) Ready mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the "Tentative Specifications for Ready Mixed Concrete" (A. S. T. M. Serial Designation: C94-35).

403—Conveying

(*a*) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

(*b*) Equipment for chuting, pumping and pneumatically conveying concrete shall be of such size and design as to insure a practically continuous flow of concrete at the delivery end without separation of the materials.

404—Depositing

(*a*) Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. No concrete that has partially hardened or been contaminated by foreign material shall be deposited on the work, nor shall retempered concrete be used.

(*b*) When concreting is once started, it shall be carried on as a continuous operation until the placing of the panel or section is completed. The top surface shall be generally level. When construction joints are necessary, they shall be made in accordance with Section 508.

(*c*) All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around reinforcement, embedded fixtures and into the corners of the forms.

(*d*) Where conditions make compacting difficult, or where the reinforcement is congested, batches of mortar containing the same proportions of cement to sand as used in the concrete, shall first be deposited in the forms. The concreting shall be carried on at such a rate that the concrete is at all times plastic and flows readily into the spaces between the bars.

405—Curing

(*a*) In all concrete structures, provision shall be made for maintaining the concrete in a moist condition for a period of at least seven days after the placement of the concrete, and for high early strength concretes, special moist curing shall be provided for at least the first three days of the seven-day period after the placement of the concrete.

406—Cold Weather Requirements

(*a*) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(*b*) All concrete materials and all reinforcement, forms, fillers and ground with which the concrete is to come in contact, shall be free from frost. Whenever the temperature of the surrounding air is below 40 degrees Fahrenheit all concrete placed in the forms shall have a temperature of between 70 degrees Fahrenheit and 100 degrees Fahrenheit, and adequate means shall be provided for maintaining a temperature of 50 degrees Fahrenheit for not less than 72 hours after placing or for as much more time as is necessary to insure proper curing of the concrete. No dependence shall be placed on salt or other chemicals for the prevention of freezing. Manure, when used for protection shall not be applied directly to concrete.

CHAPTER 5—FORMS AND DETAILS OF CONSTRUCTION

501—Design of Forms

(*a*) Forms shall conform to the shape, lines, and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

502—Removal of Forms

(*a*) Forms shall be removed in such a manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, the removable floor forms, beams and girder sides, column and similar vertical forms may be removed after 24 hours, providing the concrete is sufficiently hard not to be injured thereby. In no case shall the supporting forms or shoring be removed until the members have acquired sufficient strength to support safely their weight and the load thereon.

503—Pipes, Conduits, etc. Embedded in Concrete

(*a*) Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete except in passing through from one side to the other of a floor, wall or beam. Electric conduits and other pipes whose embedment is allowed shall not, with their

fittings, displace that concrete of a column on which stress is calculated or which is required for fire protection, to greater extent than four per cent of the area of the cross section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced concrete, provided they are not exposed to rusting or other deterioration, are of uncoated iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over two inches, and are spaced not less than three diameters on centers. Embedded pipes or conduits other than those merely passing through, shall not be larger in outside diameter than one-third the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than three diameters on centers, nor so located as unduly to impair the strength of the construction. Circular uncoated or galvanized electric conduit of iron or steel may be considered as replacing the displaced concrete.

504—Cleaning and Bending Reinforcement

(*a*) Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for stirrups and ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. Bends for other bars shall be made around a pin having a diameter not less than six times the minimum thickness of the bar, except that for bars larger than one inch, the pin shall be not less than eight times the minimum thickness of the bar. All bars shall be bent cold.

505—Placing Reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs and spacers. The minimum clear distance between parallel bars shall be 1½ times the diameter for round bars and twice the side dimension for square bars. If special anchorage as required in Sec. 903 is provided, the minimum clear spacing between parallel bars shall be equal to the diameter for round bars and 1½ times the side dimension for square bars. In no case shall the clear spacing between bars be less than 1 in., nor less than 1⅓ times the maximum size of the coarse aggregate.

(*b*) When wire or other reinforcement, not exceeding ¼ inch in diameter is used as reinforcement for slabs not exceeding ten feet in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at mid-span; provided such reinforcement is either continuous over, or securely anchored to the support.

506—Splices and Offsets in Reinforcement

(*a*) In slabs, beams and girders, splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficent lap to transfer the stress between bars by bond and shear. In such splices the minimum spacing of bars shall be as specified in Sec. 505.

(*b*) Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion shall not be more than 1 in 6, and in the case of tied columns the ties shall be spaced not over three inches on centers for a distance of one foot below the actual point of offset.

507—Concrete Protection for Reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than 3 inches of con-

crete between it and the ground contact surface. At other surfaces where concrete may be exposed to the ground or to weather, but is placed in forms, the metal reinforcement shall be protected by at least 2 inches of concrete.

(*b*) The protective covering for metal reinforcement at surfaces not exposed directly to ground or weather shall be ¾ inches for slabs and walls; 1½ inches for beams and girders; and 1½ inches for columns. In concrete joist floors in which the clear distance between joists is not more than 30 inches, the protection of metal reinforcement shall be at least ¾ inches.

(*c*) The thicknesses given in (*b*) are specified as giving normal resistance to fire. If thicknesses greater than these are required for additional fire protection by the provisions of the section on fire resistive construction of the code of which these regulations form a part, such thicknesses shall be used in lieu of those specified herein.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

508—Construction Joints

(*a*) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a joint is to be made, the surface of the concrete shall be thoroughly cleaned and all laitance removed. In addition to the foregoing, vertical joints shall be thoroughly wetted but not saturated, and slushed with a coat of neat cement grout immediately before the placing of new concrete.

(*b*) At least 2 hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(*c*) Construction joints in floors shall be located near the middle of the spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

CHAPTER 6—DESIGN—GENERAL CONSIDERATIONS

601—Assumptions

(*a*) The design of reinforced concrete members shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending. The following assumptions shall be made:

(1) The steel takes all the tensile stress.

(2) In determining the ratio n for design purposes, the modulus of elasticity for the concrete shall be taken as 1000 f'_c, and that for steel as 30,000,000 lb. per square inch.

602—Design Loads

(*a*) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

603—Resistance to Wind Forces

(*a*) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(*b*) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead- and live-loads.

(*c*) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the allowable stresses for combined live- and dead-loads provided in Sections 305 and 306 by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

CHAPTER 7—FLEXURAL COMPUTATIONS

701—General Requirements

(*a*) All members shall be designed to resist at all sections the maximum bending moments and shears produced by dead load, live load and wind load, as determined by the principle of continuity.[1]

702—Conditions of Design

(*a*) *Permissible assumptions*

The span length of freely supported beams and slabs shall be the clear span plus the depth of beam or slab, but shall not exceed the distance between centers of the supports.

In the application of the principle of continuity, the following assumptions shall be permissible:

(1) Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans.

(2) Any reasonable and consistent assumptions may be made as to the relative stiffness of the floor construction and columns. In computing the relative stiffness of floors to columns, the value I of the floor members may be based on the

[1]In the case of approximately equal spans with loads uniformly distributed, where the intensity of live load does not exceed three times the intensity of dead load, the clause is satisfied essentially by the following moments:

Negative moment at face of first interior support

For beams and girders and for slabs exceeding 10 feet

Two spans	$\frac{1}{8} wl'^2$
More than two spans	$\frac{1}{10} wl'^2$

For slabs not exceeding 10 feet in span

Two spans	$\frac{1}{10} wl'^2$
More than two spans	$\frac{1}{12} wl'^2$

Negative moment at face of other interior supports $\frac{1}{12} wl'^2$

Positive moment at center of span

End spans	$\frac{1}{10} wl'^2$
Interior spans	$\frac{1}{12} wl'^2$

Shear in end members at first interior support $1.20 \frac{wl'}{2}$

Shear at other supports $\frac{wl'}{2}$

For the purposes of applying this method "approximately" shall be construed to mean that the longer of two adjacent spans shall not exceed the shorter by more than 20 per cent. In these expressions l' = the clear span for positive moments and the average of the two adjacent clear spans for negative moment.

entire concrete sections neglecting the reinforcement, and that of columns on the entire concrete section plus the transformed steel section. The moment of inertia assumed for the columns in computing bending moments must also be used in computing stresses.

(3) The far ends of columns above and below the floor under consideration may be considered fixed.

(4) When members are deepened near their ends by haunches they may be analyzed as members of constant section provided the minimum depth is used throughout in computing stresses due to bending; otherwise a complete analysis is required. Where members are widened near their supports the additional width may be neglected in computing moments but may be used in computing stresses.

Additional section at the end may in any case be utilized in resisting shear.

(5) Where slabs of unform thickness are built integrally with their supports the span length may be taken equal to the clear span between faces of supports and the width of support otherwise neglected.

(6) In the application of the principle of continuity, center to center distances may be used in the moment determination of all members.

Moments prevailing at the faces of support may be used to proportion the members at these sections.

(*b*) *Limitations*

(1) Wherever at any section positive reinforcement is indicated by analysis, the amount provided shall be not less than .005 $b'd$ except in slabs of uniform thickness.

(2) Not less than .005 $b'd$ of negative reinforcement shall be provided at the outer end of all members built integrally with their supports.

(3) Where analysis indicates negative reinforcement along the full length of a span, the reinforcement need not be extended beyond the point where the required amount is .0025 $b'd$ or less.

(4) In slabs of unform thickness the minimum amount of reinforcement in the direction of principal stress shall be

For structural, intermediate and hard grades and rail steel......... .0025 bd
For steel having a minimum yield point of 56000 lb. per sq. in...... .002 bd

703—Depth of Beam or Slab

(*a*) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face of the structural members. Any floor finish not placed monolithically with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithically with the structural slab in buildings of the warehouse or industrial class, there shall be placed an additional depth of ½ inch over that required by the design of the member.

704—Distance between Lateral Supports

(*a*) The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange.

705—Requirements for T-Beams

(*a*) In T-beam construction the slab and beam shall be built integrally or otherwise effectively bonded together. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(*b*) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(*c*) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a rib in ribbed floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, nor in any case 18 inches.

(*d*) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Sec. 505 relating to the spacing of bars, and 404 (d), relating to the placing of concrete shall be fully met. In no case shall the area of steel in compression at any cross-section adjacent to the support exceed 2 per cent of the cross-sectional area of the stem of the beam in that section.

(*e*) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(*f*) Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

706—One-way Ribbed Floor Construction

(*a*) Ribbed floor construction consists of concrete ribs and slabs placed monolithically with or without burned clay or concrete tile fillers. The ribs shall not be farther apart than 30 inches face to face. The ribs shall be straight, not less than 4 inches wide, nor of a depth more than 3 times the width.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the ribs are used, and the fillers are so placed that the joints in alternate rows are staggered, the shells of the fillers in contact with the ribs may be included in the calculations involving shear or negative bending moment. No other portion of the fillers may be included in the design calculations.

(*c*) The concrete slab over the fillers shall be not less than 1½ inches in thickness, nor less in thickness than one-twelfth of the clear distance between ribs. Shrinkage reinforcement in the slab shall be provided as required in Sec. 708.

(*d*) Where removable forms or fillers not complying with (b) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between ribs and in no case less than two inches. Such slab shall be reinforced at right angles to the ribs with a minimum of .049 sq. in. of reinforcing steel per foot of width, and in slabs on which the prescribed live load does not exceed fifty lb. per sq.ft., no additional reinforcement will be required.

(*e*) When the finish used as a wearing surface is placed monolithically with the structural slab in buildings of the warehouse or industrial class, the thickness of the concrete over the fillers shall be ½ inch greater than the thickness used for design purposes.

(*f*) Where the slab contains conduits or pipes, the thickness shall not be less than 1 inch plus the total over-all depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to impair the strength of the construction.

707—Compression Steel in Flexural Members

(*a*) Where it is necessary to introduce steel in compression in girders, beams, or slabs, such steel shall be thoroughly anchored by ties or stirrups not less than ¼ inch

in size which shall be spaced not more than 8 inches apart over the distance where the compression steel is required.

708—Shrinkage and Temperature Reinforcement

(*a*) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area (*bd*), but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than 18 inches:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 inches	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 inches	0.0022

709—Floors with Supports on Four Sides[2]

(*a*) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete ribs with burned clay or concrete tile fillers, with or without concrete top slabs; and concrete ribs with top slabs placed monolithically with the ribs. The supports for the floor slabs may be walls, reinforced concrete beams, or steel beams fully encased in concrete.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the ribs are used, the shells in contact with the concrete ribs may be included in calculations involving resistance to shear and bending moment, and the top and bottom shells may be included in calculations involving resistance to bending moment.

(*c*) When a concrete top slab, placed monolithically with the ribs is used, it shall be not less in thickness than 1½ inches nor less than one-twelfth of the clear distance between ribs. It shall be reinforced for shrinkage as required in Section 708.

(*d*) Where removable forms or fillers not complying with 709 (b) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between ribs and in no case less than two inches. Such slab shall be reinforced to provide sufficient strength to carry the imposed loads.

(*e*) The values of the factors to be used in computations are as follows:

(1) For simple spans

$$F_A = F_B = 1$$

(2) For end spans, continuous at one end only

$$F_A = 1 - \left[\frac{.25}{1 + \frac{7}{8}\frac{K_A}{K_{AR}}}\right] \qquad (2)$$

$$F_B = 1 - \left[\frac{.25}{1 + \frac{7}{8}\frac{K_B}{K_{BR}}}\right] \qquad (3)$$

[2]No exact theoretical solution of the problem of the stresses in a series of adjacent unequal rectangular panels has yet been brought to the attention of the committee. The approximate solution embodied in this Section is believed to be conservative, but the best available.

(3) For interior continuous spans

$$F_A = \frac{1}{2}\sqrt{1 - U_{AR}} + \frac{1}{2}\sqrt{1 - U_{AL}} \quad \ldots\ldots (4)$$

$$F_B = \frac{1}{2}\sqrt{1 - U_{BR}} + \frac{1}{2}\sqrt{1 - U_{BL}} \quad \ldots\ldots (5)$$

in which

$$U_{AR} = \frac{1}{1.5 + \frac{7}{8}\frac{K_A}{K_{AR}}}$$

and

$$U_{AL} = \frac{1}{1.5 + \frac{7}{8}\frac{K_A}{K_{AL}}}$$

(4) For strips rigidly anchored to the supports

$$e_A = \frac{2}{4 - D_B} \text{ (for span } A) \quad \ldots\ldots (6)$$

$$e_B = \frac{2}{4 - D_A} \text{ for span } B) \quad \ldots\ldots (6a)$$

in which

$$D_A = \frac{F_A A}{F_B B}$$

$$D_B = \frac{F_B B}{F_A A}$$

(5) For strips not rigidly anchored at one or both ends, and for ribbed construction without filler blocks

$e_A = 1.0$ (for span A)
$e_B = 1.0$ (for span B)

(6) Load distribution factor

$$r_A = \frac{1}{1 + D_A{}^3} \text{ (for span } A) \quad \ldots\ldots (7)$$

$$r_B = 1 - r_A \text{ (for span } B) \quad \ldots\ldots (7a)$$

(*f*) *Slab Thickness*

The minimum thickness of the slab which shall not be less than 4 inches shall be computed by formula (8).

$$t_3 = \frac{A + B - 0.10\,N}{72}\sqrt[3]{\frac{2000}{f'_c}} \quad \ldots\ldots (8)$$

(*g*) *Bending Moments*

(1) The bending moments at any section of any strip one foot wide, extending the full length of the continuous slab, shall be determined on the basis of the recognized principles of mechanics relating to continuous beams for those conditions of load-

ing which cause maximum moment at any section, using an equivalent uniform load of $e_A r_A w$ in the A direction, and $e_B r_B w$ in the B direction.

(*h*) *Supporting Beams*

(1) For span A, the maximum bending moments in the supporting beams may be determined from an equivalent uniformly distributed load per linear foot of

$$M_{max} = w \frac{B}{2} (1 - e_A r_A) \qquad (9)$$

(2) For span A, the maximum shear in the supporting beams may be determined by formula (12).

$$\text{Shear}_{max} = w \frac{BA}{4} (1 - r_A) \qquad (10)$$

(3) For the purpose of computing shear and bending moments at intermediate points for the supporting beams of span A, the total load from a two-way panel AB, carried on the beam of span A shall at least be equal to

Total Load $= w\dfrac{BA}{2} (1 - r_A)$, considered uniformly varying in intensity

from $w\dfrac{B}{2} (1 + 2r_A - 3e_A r_A)$ at the center .... (11)

to $w\dfrac{B}{2} (1 - 4r_A + 3e_A r_A)$ at the supports .... (11a)

(4) For span B, in the foregoing formulas, replace A with B, B with A, r_A with r_B, and e_A with e_B.

(*i*) *Panels of Approximately Uniform Stiffness*

When the ratio of the stiffness factor of the span under consideration to that of each adjacent span is at least ⅔ and at most 3/2, F_A, or F_B, may be taken as 0.76 for interior spans, 0.87 for end spans and 1.0 for simple spans.

(*j*) *Shear in Slab*

(1) For purposes of determining shear in the strip one foot wide carrying the maximum load, the total load for the length of span A shall be

Total Load $= r_A w A$, considered uniformly varying in intensity from
$r_A w\ (3e_A - 2)$ at the center of the span to .... (12)
$r_A w\ (4 - 3e_A)$ at the supports .... (12a)

(2) Similarly, the total load carried on a strip one foot wide for the length of span B shall be

Total Load $= r_B w B$, considered uniformly varying in intensity from
$r_B w\ (3e_B - 2)$ at the center of the span, to .... (13)
$r_B w\ (4 - 3e_B)$ at the supports .... (13a)

(*k*) *Arrangement of Reinforcement*

(1) In any panel, the reinforcement per unit width in the long direction shall be at least one-third of that provided in the short direction.

(2) The positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.

(3) At a non-continuous edge negative moment reinforcement per unit width in amount at least as great as one-half of that required for maximum positive moment for the center one-half of the panel shall be provided across the entire width of the exterior support.

(4) The spacing of the reinforcement shall be not more than 3 times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

(5) The amount of reinforcement in any one foot wide strip shall be at least 0.003 times the product of the width of strip by the effective depth.

710—Maximum Spacing of Principal Slab Reinforcement

(*a*) In slabs other than ribbed floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 708 (a).

CHAPTER 8—SHEAR AND DIAGONAL TENSION

801—Shearing Unit Stress

(*a*) The shearing unit stress (v) in reinforced concrete beams shall be computed by formula (14):

$$v = \frac{V}{bjd} \quad \text{.......... (14)}$$

(*b*) For beams of I or T section b' shall be substituted for b in formula (14).

(*c*) In ribbed construction, where burned clay or concrete tile are used, b' may be taken as a width equal to the thickness of the concrete web plus the thicknesses of the vertical shells of the concrete or burned clay tile in contact with the joist as in Sec. 707 (b).

(*d*) When the value of the shearing unit stress computed by formula (14) exceeds the shearing unit stress (v_c) permitted on the concrete of an unreinforced web (see Sec. 305 (a)), web reinforcement shall be provided to carry the excess.

802—Types of Web Reinforcement

(*a*) Web reinforcement may consist of:

(1) Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

(2) Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30 degrees or more thereto.

(3) Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.

(4) Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the concrete by the reinforcement (see Sec. 804(f).)

(*b*) Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Sec. 904.

803—Stirrups

(*a*) The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by formula (15).

$$A_v = \frac{V's}{f_v\, jd} \quad \text{.......... (15)}$$

(*b*) Inclined stirrups shall be proportioned by formula (17) (Sec. 804 d).

(*c*) Stirrups placed perpendicular to the longitudinal reinforcement shall not be used alone as web reinforcement when the shearing unit stress (v) exceeds $0.08f'_c$.

804—Bent Bars

(*a*) When the web reinforcement consists of a single bent bar or of a single group of bent bars the required area of such bars shall be computed by formula (16).

$$A_v = \frac{V'}{f_v \sin \alpha} \quad \text{.......... (16)}$$

(*b*) In formula (16) V' shall not exceed $0.040\, f'_c\, bjd$.

(*c*) Only the center three-fourths of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

(*d*) Where there is a series of parallel bent bars, the required area shall be determined by formula (17).

$$A_v = \frac{V's}{f_v jd\ (sin\ \alpha + cos\ \alpha)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

(*e*) When bent bars, having a radius of bend of not more than two times the diameter of the bar are used alone as web reinforcement, the allowable shearing unit stress shall not exceed 0.060 f'_c. This shearing unit stress may be increased at the rate of 0.01 f'_c for each increase of four bar diameters in the radius of bend until the maximum allowable shearing unit stress is reached. (See Sec. 305 (a).)

(*f*) The shearing unit stress permitted when special arrangements of bars are employed shall be that determined by making comparative tests, to destruction, of specimens of the proposed system and of similar specimens reinforced in conformity with the provisions of this code, the same factor of safety being applied in both cases.

805—Combined Web Reinforcement

(*a*) Where more than one type of reinforcement is used to reinforce the same portion of the web, the total shearing resistance of this portion of the web shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once, and no one type of reinforcement shall be assumed to resist more than $\frac{2\ V'}{3}$.

806—Spacing of Web Reinforcement

(*a*) Where web reinforcement is required it shall be so spaced that every 45 degree line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement. If a shearing unit stress in excess of 0.06 f'_c is used, every such line shall be crossed by at least two such lines of web reinforcement.

807—Shearing Stress in Flat Slabs

(*a*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$ in. beyond the edge of the column capital and parallel with it, shall not exceed the following values when computed by formula (14) (in which d shall be taken as $t_1 - 1\frac{1}{2}$ in.):

(1) 0.03 f'_c, when at least 50 per cent of the total negative reinforcement passes directly over the column capital.

(2) 0.025 f'_c, when 25 per cent of the total negative reinforcement passes directly over the column capital.

(3) For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(*b*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_2 - 1\frac{1}{2}$ in. beyond the edge of the dropped panel and parallel with it shall not exceed 0.03 f'_c when computed by formula (14) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in the column strip must be within the width of strip directly above the dropped panel.

808—Shear and Diagonal Tension in Footings

(*a*) The shearing unit stress computed by formula (14) on the critical section (see 1204 (a)), shall not exceed 0.02 f'_c for footings with straight bars, nor 0.03 f'_c for footings in which the bars are anchored at both ends by adequate hooks or as otherwise specified in Sec. 905.

CHAPTER 9—BOND AND ANCHORAGE

901—Computation of Bond Stress in Beams

(*a*) In flexural members in which the tensile reinforcement is parallel to the compression face, the bond stress at any cross section shall be computed by formula (18).

$$u = \frac{V}{\Sigma 0 \; jd} \qquad (18)$$

in which V is the shear at that section.

(*b*) Adequate end anchorage shall be provided for the tensile reinforcement in all flexural members to which formula (18) does not apply, such as footings, brackets and other tapered or stepped beams in which the tensile reinforcement is not parallel to the compression face.

902—Ordinary Anchorage Requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar shall be extended at least 12 diameters beyond the point at which it is no longer needed to resist stress. In cases where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond, the bar shall extend into a region of compression and be anchored by means of a standard hook or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and either made continuous with the positive reinforcement or anchored in a region of compression.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of ten or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

(*c*) In simple beams, or at the outer ends of freely supported end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of ten or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

903—Special Anchorage Requirements

(*a*) Where increased shearing or bond stresses are permitted because of the use of special anchorage (see Sec. 305), every bar shall be terminated in a standard hook in a region of compression, or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the negative or positive reinforcement.

904—Anchorage of Web Reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

(1) Welding to longitudinal reinforcement.

(2) Hooking tightly around the longitudinal reinforcement through 180 degrees.

(3) Embedment in the compression area of the beam a distance sufficient to develop the allowable tensile stress specified in Sec. 306 at a bond stress not exceeding $.04 f'_c$ on plain bars nor $.05 f'_c$ on deformed bars.

(4) Standard hook plus embedment in the compression area of the beam, which embedment exclusive of the hook shall be sufficient to develop by bond a stress

of not less than 10,000 lb. per sq. in. at a bond stress of not more than .04 f'_c on plain bars nor .05 f'_c on deformed bars.

(*b*) The extreme ends of bars forming simple *U* or multiple stirrups shall be anchored by one of the methods of Sec. 904 (a) or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 diameters of the stirrup bar.

(*c*) The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees, or by being welded or otherwise rigidly attached thereto.

(*d*) Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

(1) As specified in Sec. 904 (a), (3).

(2) As specified in Sec. 904 (a), (4).

(3) By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam, plus an extension of the bar of not less than 12 diameters of the bar terminating in a standard hook. This short radius bend extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 lb. per sq. in.

(4) By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend, of radius not less than 2 times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 lb. per sq. in.

(5) The tensile unit stress at the beginning of a bend may be increased from 10,000 lb. per sq. in. when the radius of bend is 2 bar diameters, at the rate of 1,000 lb. per sq. in. tension for each increase of 1½ bar diameters in the radius of bend, provided that the length of the bar in the bend and extension is sufficient to develop this increased tensile stress by bond at the unit stresses given in Sec. 904 (e) (3).

(*f*) In all cases web reinforcement shall be carried as close to the compression surface of the beam as fireproofing regulations and the proximity of other steel will permit.

905—Anchorage of Bars in Footing Slabs

(*a*) All bars in footing slabs, except the longitudinal reinforcement between loads in continuous slab footings, shall be anchored by means of standard hooks. The outer faces of these hooks shall be not less than three inches nor more than six inches from the face of the footing.

906—Hooks

(*a*) The terms "hook" or "standard hook" as used herein shall mean a complete semicircular turn with a radius of bend on the axis of the bar of not less than three and not more than six bar diameters, plus an extension of at least four bar diameters

at the free end of the bar. Hooks having a radius of bend of more than six bar diameters shall be considered merely as extensions to the bars, and shall be treated as in Sec. 904 (e) (5).

(*b*) In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(*c*) No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 10,000 lb. per sq. in.

(*d*) Any mechanical device capable of developing the strength of the bar without damage to the concrete may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

CHAPTER 10—FLAT SLABS—TWO-WAY AND FOUR-WAY SYSTEMS WITH SQUARE OR RECTANGULAR PANELS

1001—Limitations

(*a*) The term flat slabs as used in these regulations refers to concrete slabs, without beams or girders to carry the load to supporting members, reinforced with bars extending in two or four directions. Slabs with dropped panels or paneled ceilings shall be considered as flat slabs provided that they meet the requirements herein given for such construction.

(*b*) The moment coefficients, moment distribution, and slab thicknesses specified herein are for a series of rectangular slabs of approximately uniform size arranged in three or more rows of panels in each direction, and in which the ratio of length to width of panel does not exceed 1.33.

(*c*) For structures having a width of less than three rows of panels, or in which irregular panels are used, an analysis shall be made of the moments developed in both slabs and columns.* When so required, computations shall be submitted to the Commissioner of Buildings for approval.

1002—Panel Strips and Principal Design Sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one-half panel in width, symmetrical about panel center line and extending through the panel in the direction in which moments are considered.

A column strip one-half panel in width occupying the two-quarter panel areas outside of the middle strip.

(*b*) The critical sections for moment calculations are referred to as principal design sections and are located as follows:

Sections for Negative Moment. These shall be taken along the edges of the panel, on lines joining the column centers, except that they follow the perimeter of the column capital instead of passing through it.

Sections for Positive Moment. These shall be taken on the center lines of the panel.

(*c*) In the two-way system it shall be assumed that the various moments in the strips are resisted by the bands located within the strips, each band being .5 l_1, in width.

(*d*) In the four-way system, it shall be assumed that the column strip positive moment is resisted by the direct band; that the column strip negative moment is

*It is not the intention to prohibit flat slab construction for panels longer than 1.33 times the width, or for buildings less than three bays wide, provided the moment factors are properly adjusted.

resisted by the direct band plus the two diagonal bands multiplied by the cosine of the angle between the direct band and the diagonal bands; that the middle strip positive moment is resisted by the two diagonal bands multiplied by the cosine of the angle between the axis of the middle strip and the diagonal bands; and that the middle strip negative moment is resisted by an independent top band across the middle of the direct band. The width of direct and middle strip negative bands shall be approximately .4 l_1, the width of the diagonal bands shall be approximately .4 of the average span length or

$$\frac{(l + l_1)}{5}$$

(*e*) The width of the column head section for compression shall be taken as the width of the dropped panel (b_1), or half the width of the panel (.5 l_1) where no dropped panel is used.

1003—Slab Thickness and Dropped Panel Sizes

(*a*) In Table 1 are given the thicknesses, dimensions and moments governing flat slab design when f'_c equals 2,000 lb. per sq. in. The general formulas are given under the heading "General Case;" the formulas for the case where the diameter of the column capital (c) = 0.225 l are given under the heading "Special Case" for (c) = 0.225 l.

(*b*) Where f'_c is greater than 2,000 lb. per sq. in., the required and minimum slab thicknesses given in Table 1 may be reduced by multiplying by the factor

$$\sqrt[3]{\frac{2000,}{f'_c}}$$

in which f'_c is the ultimate 28-day compressive strength of the concrete to be used.

TABLE 1—LIMITATIONS FOR SLAB THICKNESSES, DROPPED PANELS AND MOMENTS

	Symbol	Unit	General Case	Special Case For $c = .225l$
Minimum Floor Slab Thickness	t_1 or t_2	Inches	.375 (long l)	0.375 (long l)
Minimum Roof Slab Thickness	t_1 or t_2	Inches	0.300 (long l)	0.300 (long l)
Slab Thickness without Dropped Panel	t_1	Inches	$0.038\left(1 - 1.44\frac{c}{l}\right) l \sqrt{w'} + 1\frac{1}{2}$ (19)	$0.025l \sqrt{w'} + 1\frac{1}{2}$ (19a)
Slab Thickness beyond Dropped Panel	t_2	Inches	$0.02l \sqrt{w'} + 1$ (20)	$0.02l \sqrt{w'} + 1$ (20a)
Slab Thickness through Dropped Panel	t_1	Inches	(Minimum = 1.25 t_2 (Maximum = 1.50 t_2	(Minimum = 1.25 t_2 (Maximum = 1.50 t_2
Maximum l to be used in thickness formulas				
Minimum side or diameter of Dropped Panel	b_1	Feet	$0.35l_1$	$0.35l_1$
Numerical sum of positive and negative moments in direction of either side of interior rectangular panel	M_o	ft. lb.	$0.09Wl\left(1 - \frac{2c}{3l}\right)^2$ (21)	$0.065Wl$ (21a)

In these tables (l), (l_1), (b_1) and (c) are always expressed in feet while the units to which the formulas develop are shown in the column headed "units."

TABLE 2—MOMENTS TO BE USED IN DESIGN OF AN INTERIOR PANEL OF FLAT SLAB

	Symbol	Units	General Case	Special Case $c = .225l$
TWO-WAY SYSTEM WITH DROPPED PANEL				
Column Strip, Negative Moment	$-M_c$	ft. lb.	$0.50M_o$	$0.0325Wl$
Column Strip, Positive Moment	$+M_c$	ft. lb.	$0.20M_o$	$0.0130Wl$
Middle Strip, Negative Moment	$-M_m$	ft. lb.	$0.15M_o$	$0.00975Wl$
Middle Strip, Positive Moment	$+M_m$	ft. lb.	$0.15M_o$	$0.00975Wl$
TWO-WAY SYSTEM WITHOUT DROPPED PANEL				
Column Strip, Negative Moment	$-M_c$	ft. lb.	$0.46M_o$	$0.030Wl$
Column Strip, Positive Moment	$+M_c$	ft. lb.	$0.22M_o$	$0.0142Wl$
Middle Strip, Negative Moment	$-M_m$	ft. lb.	$0.16M_o$	$0.0104Wl$
Middle Strip, Positive Moment	$+M_m$	ft. lb.	$0.16M_o$	$0.0104Wl$
FOUR-WAY SYSTEM WITH DROPPED PANELS (MOMENTS BY STRIPS)				
Column Strip, Negative Moment	$-M_c$	ft. lb.	$0.54M_o$	$0.0351Wl$
Column Strip, Positive Moment	$+M_c$	ft. lb.	$0.19M_o$	$0.0124Wl$
Middle Strip, Negative Moment	$-M_m$	ft. lb.	$0.08M_o$	$0.0052Wl$
Middle Strip, Positive Moment	$+M_m$	ft. lb.	$0.19M_o$	$0.0124Wl$
(MOMENTS BY BANDS)				
Direct Band, Negative Moment	$-M$	ft. lb.	$0.307M_o$	$0.0200Wl$
Direct Band, Positive Moment	$+M$	ft. lb.	$0.19M_o$	$0.0124Wl$
Diagonal Band, Negative Moment	$-M$	ft. lb.	$0.168M_o$	$0.0109Wl$
Diagonal Band, Positive Moment	$+M$	ft. lb.	$0.134M_o$	$0.0087Wl$
Cross Band, Negative Moment	$-M$	ft. lb.	$0.08M_o$	$0.0052Wl$
FOUR-WAY SYSTEM WITHOUT DROPPED PANELS (MOMENTS BY STRIPS)				
Column Strip, Negative Moment	$-M$	ft. lb.	$0.50M_o$	$0.0325Wl$
Column Strip, Positive Moment	$+M$	ft. lb.	$0.20M_o$	$0.0130Wl$
Middle Strip, Negative Moment	$-M$	ft. lb.	$0.10M_o$	$0.0065Wl$
Middle Strip, Positive Moment	$+M$	ft. lb.	$0.20M_o$	$0.0130Wl$
(MOMENTS BY BANDS)				
Direct Band, Negative Moment	$-M$	ft. lb.	$0.30M_o$	$0.0195Wl$
Direct Band, Positive Moment	$+M$	ft. lb.	$0.20M_o$	$0.0130Wl$
Diagonal Band, Negative Moment	$-M$	ft. lb.	$0.141M_o$	$0.0092Wl$
Diagonal Band, Positive Moment	$+M$	ft. lb.	$0.141M_o$	$0.0092Wl$
Cross Band, Negative Moment	$-M$	ft. lb.	$0.10M_o$	$0.0065Wl$

1004—Column Capital Sizes

(a) The average (c) for the columns at the four corners of a panel shall be used in obtaining the slab thickness, the numerical sum of the total positive and negative moments (M_o) in either direction and the middle strip positive and negative moments in either direction.

(b) The average (c) for two adjacent columns shall be used in obtaining the positive and negative moments in the column strip between these adjacent columns.

1005—Panels with Marginal Beams or Reinforced Bearing Walls

1006—Limitations (Applicable to Tables 2, 3 and 4)

(a) Any of the above moments may be varied by not more than six per cent, provided that the total numerical sum of the positive and negative moments on the principal design sections is not reduced.

(b) The ratio of reinforcement considered in any strip shall not exceed the value of (p) calculated for balanced reinforcement by Sec. 305, 306. The ratio of reinforcement in any strip shall not be less than .0025. Bars shall not be spaced farther apart than 1½ times the slab thickness for the full width of the bands.

TABLE 3—MOMENTS TO BE USED IN DESIGN OF AN EXTERIOR PANEL OF FLAT SLAB

Moments in the strips perpendicular to the discontinuous edge where they differ from an interior panel, are given in the following table. Negative moments in the column strip and middle strip on the line of the first interior columns are the same as for an interior panel. Moments in the strips parallel to the discontinuous edge are the same as for an interior panel.

	Symbol	Units	General Case	Special Case $c = .225l$
TWO-WAY SYSTEM WITH DROPPED PANEL				
Column Strip Negative Moment at discontinuous edge	$-M_c$	ft. lb.	$0.45M_o$	$0.029Wl$
Column Strip Positive Moment	$+M_c$	ft. lb.	$0.25M_o$	$0.016Wl$
Middle Strip Negative Moment at discontinuous edge	$-M_m$	ft. lb.	$0.10M_o$	$0.0065Wl$
Middle Strip Positive Moment	$+M_m$	ft. lb.	$0.19M_o$	$0.012Wl$
TWO-WAY SYSTEM WITHOUT DROPPED PANEL				
Column Strip Negative Moment at discontinuous edge	$-M_c$	ft. lb.	$0.41M_o$	$0.027Wl$
Column Strip Positive Moment	$+M_c$	ft. lb.	$0.28M_o$	$0.018Wl$
Middle Strip Negative Moment at discontintuous edge	$-M_m$	ft. lb.	$0.10M_o$	$0.007Wl$
Middle Strip Positive Moment	$+M_m$	ft. lb.	$0.20M_o$	$0.013Wl$
FOUR-WAY SYSTEM WITH DROPPED PANELS (MOMENTS BY STRIPS)				
Column Strip Negative Moment at discontinuous edge	$-M$	ft. lb.	$0.485M_o$	$0.0315Wl$
Column Strip Positive Moment	$+M$	ft. lb.	$0.24M_o$	$0.0156Wl$
Middle Strip Negative Moment at discontinuous edge	$-M$	ft. lb.	$0.05M_o$	$0.0032Wl$
Middle Strip Positive Moment	$+M$	ft. lb.	$0.24M_o$	$0.0156Wl$
(MOMENTS BY BANDS) (FOR SQUARE PANEL)				
Direct Band at Column Head at discontinuous edge	$-M$	ft. lb.	$0.28M_o$	$0.018Wl$
Direct Band at Center	$+M$	ft. lb.	$0.24M_o$	$0.0156Wl$
Diagonal Bands at Column Head at discontinuous edge	$-M$	ft. lb.	$0.15M_o$	$0.010Wl$
Diagonal Bands at Center	$+M$	ft. lb.	$0.17M_o$	$0.011Wl$
Top Band (across Middle of Direct) at discontinuous edge	$-M$	ft. lb.	$0.05M_o$	$0.003Wl$
FOUR-WAY SYSTEM WITHOUT DROPPED PANELS (MOMENTS BY STRIPS)				
Column Strip Negative Moment at discontinuous edge	$-M$	ft. lb.	$0.45M_o$	$0.029Wl$
Column Strip Positive Moment	$+M$	ft. lb.	$0.25M_o$	$0.0163Wl$
Middle Strip Negative Moment at discontinuous edge	$-M$	ft. lb.	$0.062M_o$	$0.004Wl$
Middle Strip Positive Moment	$+M$	ft. lb.	$0.25M_o$	$0.016Wl$
(MOMENTS BY BANDS) (FOR SQUARE PANEL)				
Direct Band at Column Head at discontinuous edge	$-M$	ft. lb.	$0.27M_o$	$0.017Wl$
Direct Band at Center	$+M$	ft. lb.	$0.25M_o$	$0.016Wl$
Diagonal Bands at Column Head at discontinuous edge	$-M$	ft. lb.	$0.13M_o$	$0.0084Wl$
Diagonal Bands at Center	$+M$	ft. lb.	$0.18M_o$	$0.0117Wl$
Top Band (across Middle of Direct) at discontinuous edge	$-M$	ft. lb.	$0.06M_o$	$0.004Wl$

(*c*) Moments for the four-way system are shown in the above table by strips, and for convenience, also by bands.

(*d*) Slabs supported by marginal beams on opposite edges shall be designed as solid one or two-way slabs to carry the entire panel load.

TABLE 4—MOMENTS TO BE USED IN DESIGN OF PANELS WITH MARGINAL BEAMS OR REINFORCED BEARING WALLS

		Marginal Beams with Depth greater than 1½ times the Slab Thickness; or reinforced Bearing Wall.					Marginal Beam with depth 1½ times the Slab Thickness or less.			
(a) Load to be carried by Marginal Beam or Wall.		Loads directly superimposed upon it plus a uniform load equal to one-quarter of the total live and dead panel load.					Loads directly superimposed upon it exclusive of any panel load.			
		Two-Way System		Four-Way System			Two-Way System		Four-Way System	
		With Drop	Without Drop	With Drop	Without Drop		With Drop	Without Drop	With Drop	Without Drop
(b) Moment to be used in the design of Half Column Strip adjacent and parallel to Marginal Beam or Wall.	Neg.	$.125M_o$	$.115M_o$	$.135M_o$	$.125M_o$	Neg.	$.25M_o$	$.23M_o$	$.27M_o$	$.25M_o$
	Pos.	$.05\ M_o$	$.055M_o$	$.0475M_o$	$.05M_o$	Pos.	$.10M_o$	$.11M_o$	$.095M_o$	$.10M_o$
(c) Negative Moment to be used in Design of Middle Strip continuous over Beam or Wall.	Neg.	$.195M_o$	$.208M_o$	$.104M_o$	$.13M_o$	Neg.	$.15M_o$	$.16M_o$	$.08M_o$	$.10M_o$

1007—Length of Bars and Points of Bend

The positive moment reinforcement perpendicular to the discontinuous edge shall extend to this edge and have an embedment of at least 6 inches in spandrel beams or columns. All negative moment reinforcement shall be bent or hooked at spandrel beams or columns to provide adequate bond resistance. Length of bars and points of bend shall be as given in Table 5.

1008—Arrangement of Reinforcement

(*a*) The slab reinforcement shall be accurately placed so as to resist not only the moments at the critical sections, but also the moments at intermediate sections, and shall be secured and supported by concrete or metal chairs and spacers.

1009—Brackets

(*a*) Brackets extending the full width of the column may be substituted for column capitals at exterior columns, provided the sloping face of the bracket makes an angle not more than forty-five degrees with the face of the column, projected upward.

(*b*) The value of (*c*) where brackets are used is twice the distance from the center of the column to a point where the bracket is 1½ inches thick.

1010—Columns Without Capitals or Brackets

(*a*) Brackets and column capitals may be omitted altogether, provided the slab thickness is sufficient to fully resist the moments and shears at the column head section.

(*b*) The value of (*c*) where brackets and column capitals are omitted is the width of the column in the direction in which moments are considered, except that, when a beam of greater depth than the thickness of the slab or dropped panel extends into

TABLE 5—LENGTH OF BARS AND POINTS OF BEND

	With Drop		Without Drop	
	General Case	$c = .225l$	General Case	$c = .225l$
TWO-WAY FLAT SLAB (COLUMN STRIP)				
Length of straight bars (not less than .4 of total band steel)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel)	$1.5l + .6c$†	$1.635l$†	$1.44l + .66c$†	$1.59l$†
Length of additional straight bars over column head (if required)	$.5l + .6c$	$.635l$	$.44l + .66c$	$.59l$
Point of top bend in bent bars (from column center)	$.25l$	$.25l$	$.25l$	$.25l$
MIDDLE STRIP				
Length of straight bars (not more than .5 total band steel)	$.65l$	$.65l$	$.7l$	$.7l$
Length of bent bars (not less than .5 total band steel)	$1.5l$†	$1.5l$†	$1.5l$†	$1.5l$†
Point of top bend in bent bars (from column centers)	$.175l$	$.175l$	$.15l$	$.15l$
FOUR-WAY FLAT SLAB COLUMN STRIP				
Length of straight bars (not less than .4 total band steel)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel)	$1.5l + .6c$†	$1.635l$†	$1.44l + .66c$†	$1.59l$†
Length of additional straight bars over column head (if required)	$.5l + .6c$	$.635l$	$.44l + .66c$	$.59l$
Point of bend for bent bars (from column centers)	$.2l$	$.2l$	$.2l$	$.2l$
DIAGONAL BAND				
Length of straight bars (not more than .6 total band steel area)	$l - b + (2'$ or $40d)$	$.65l + (2'$ or $40d)$	$.75l$	$.75l$
Length of bent bars (not less than .4 total band steel area)	$2.21l$†	$2.21l$†	$2.21l$†	$2.21l$†
Point of bend for bent bars (from column centers)	$.33l$	$.33l$	$.33l$	$.33l$
Length of additional straight bars over column head (if required)	$.8l$	$.8l$	$.8l$	$.8l$
Top band across middle of direct band (length of straight bars)	$.5l$	$.5l$	$.5l$	$.5l$

†Note: To these lengths proper allowance to be added for bends.

the column in the direction in which moments are considered, the value of (c) may be taken as the width of the column plus twice the projection of the beam below the slab or dropped panel.

1011—Openings in Flat Slabs

(a) Openings of any size may be cut through the floor in the area common to two intersecting middle strips, provided the total positive and negative resisting moments be maintained as required in Section 1004 and that these total positive and total negative moments be redistributed between the remaining principal design sections to meet the new conditions.

(b) In any area common to two column strips, not more than one opening shall be allowed and the greatest dimension of such an opening shall not exceed .05 l.

(c) In any area common to one column strip and one middle strip, openings shall not interrupt more than one-quarter of the bars in either strip and the equivalent of the bars so interrupted shall be provided by extra steel on both sides of the opening.

(*d*) Any opening larger than described above shall be completely framed on all sides with beams to carry the loads to the columns.

1012—Shearing Stresses in Flat Slabs. See Section 807

CHAPTER 11—REINFORCED CONCRETE COLUMNS AND WALLS

1101—Limiting Dimensions

(*a*) The following sections on reinforced concrete and composite columns except Section 1107 (a) apply to a short column, for which the unsupported length is not greater than ten times the least lateral dimension. When the unsupported length exceeds this value, the design shall be modified as shown in Section 1107 (a). Principal columns in buildings shall have a minimum diameter or thickness of 10 inches and a minimum gross area of 120 sq. in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 inches.

1102—Unsupported Length of Columns

(*a*) For purposes of determining the limiting dimensions of columns, the unsupported length of reinforced concrete columns shall be taken as the clear distance between floor slabs, except that

(1) In flat slab construction, it shall be the clear distance between the floor and the lower extremity of the capital.

(2) In beam and slab construction, it shall be the clear distance between the floor and the under side ot the deeper beam framing into the column in each direction at the next higher floor level.

(3) In columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane; provided that to be an adequate support, two such struts shall meet the column at approximately the same level, and the angle between vertical planes through the struts shall not vary more than 15 degrees from a right angle. Such struts shall be of adequate dimensions and anchorage to restrain the column against lateral deflection.

(4) In columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the column.

(*b*) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

1103—Spirally Reinforced Columns

(*a*) *Permissible Load*—The maximum permissible axial load, P, on columns with closely spaced spirals enclosing a circular concrete core reinforced with longitudinal bars shall be that given by Formula (22).

$$P = A_g\ (0.22\ f'_c + f_s p_g) \qquad (22)$$

Wherein A_g = the gross area of the column

f'_c = compressive strength of the concrete

f_s = nominal working stress in vertical column reinforcement, to be taken at 40 per cent of the minimum specification value of the yield point; viz., 16,000 lb. per sq. in. for intermediate grade steel and 20,000 lb. per sq. in. for rail or hard grade steel.[1]

[1]Nominal working stresses for reinforcement of higher yield point may be established at 40 per cent of the yield point stress, but not more than 30,000 lb. per sq. in., when the properties of such reinforcing steels have been definitely specified by standards of A. S. T. M. designation. If this is done, the lengths of splice required by Section 1103 (c) shall be increased accordingly.

p_g = ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.

(*b*) *Vertical Reinforcement*—The ratio (p_g) shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be six, and the minimum diameter shall be ⅝ in. The center to center spacing of bars within the periphery of the column core shall not be less than 2½ times the diameter for round bars or 3 times the side dimension for square bars. The clear spacing between bars shall not be less than 1½ inches or 1½ times the maximum size of the coarse aggregate used. These spacing rules also apply to adjacent bars at a lapped splice.

(*c*) *Splices in Vertical Reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

(1) For deformed bars—with concrete having a strength of 3000 lb. per sq. in. or above, 24 diameters of bar of intermediate grade steel and 30 diameters of bar of rail steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the nominal working stress. When the concrete strengths are less than 3000 lb. per sq. in., the amount of lap shall be one-third greater than the values given above.

(2) For plain bars—the minimum amount of lap shall be 25 per cent greater than that specified for deformed bars.

(3) Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds 1¼ in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

(4) Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded by a concrete capital, floor slab or by metal ties or reinforcing spirals. Where bars are offset, the slope of the inclined portion from the axis of the column shall not exceed 1 in 6 and the bars above and below the offset shall be parallel to the axis of the column.

(*d*) *Spiral Reinforcement*

The ratio of spiral reinforcement, (p'), shall not be less than the value given by Formula (23), nor shall it be less in any case than 0.0112 for hot rolled spirals of intermediate grade or 0.0075 for cold drawn wire spirals.

$$p' = 0.45\ (R-1)\frac{f'_c}{f'_s} \qquad \text{(23)}$$

Wherein p' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

R = ratio of gross area to core area of column, A_g/A_c.

f'_s = useful limit stress of spiral reinforcement, to be taken as 40,000 lb. per sq. in. for hot rolled rods of intermediate grade (A. S. T. M. Serial Designation: A15-35) and 60,000 lb. per sq. in. for cold drawn wire (A. S. T. M. Serial Designation: A82-34).

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral rod or wire at each end of the spiral unit. Splices, when necessary, shall be made in spiral rod or wire by welding or by a lap of 1½ turns. The center to center spacing of the spirals shall not

exceed one-sixth of the core diameter. The clear spacing between spirals shall not exceed 3 in. nor be less than 1⅜ in. or 1½ times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the floor level in any story or from the top of the footing in the basement, to the level of the lowest horizontal reinforcement in the slab, dropped panel or beam above. In a column with a capital, it shall extend to the plane at which the diameter or width of the capital is twice that of the column.

(*e*) *Protection of Reinforcement*

The column reinforcement shall be protected everywhere by a covering of concrete cast monolithically with the core, for which the thickness shall not be less than 1½ in. nor less than 1½ times the maximum size of the coarse aggregate, nor shall it be less than required by the fire protection and weathering provisions of Section 506 (b).

(*f*) *Limits of Column Section*

For columns built monolithically with concrete walls or piers, the outer boundary of the column section shall be taken either as a circle 1½ in. outside the column spiral or as a square or rectangle of which the sides are 1½ in. outside the spiral. The value of A_g thus defined shall be used in both Formulas (22) and (23). In any case it shall be permissible to design a circular column and to build it as a square column of the same least lateral dimension. In such case the permissible load, the gross area considered, and the required percentage of reinforcement must be taken as those of the circular column.

1104—Tied Columns

(*a*) *Permissible Load*—The maximum permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 70 per cent of that given by Formula (22). The ratio, (p_g), to be considered in tied columns shall not be less than 0.01 nor more than 0.04. The longitudinal reinforcement shall consist of at least four bars, of minimum diameter of ⅝ inch. Splices in reinforcing bars shall be made as described in Section 1103 (c).

(*b*) *Lateral Ties*—Lateral ties shall be at least ¼ in. in diameter and shall be spaced apart not over 16 bar diameters, 48 tie diameters or the least dimension of the column. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar is held firmly in its designed position and has lateral support equivalent to that provided by a 90-degree corner of a tie.

(*c*) *Limits of Column Section*

In a tied column which for architectural reasons has a larger cross section than required by considerations of loading, a reduced effective area (A_g) not less than one-half of the total area may be used in applying the provisions of Section 1104 (a).

1105—Composite Columns

(*a*) *Permissible Load* The permissible load on a composite column consisting of a structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by Formula (24).

$$P = 0.22\, A_c f'_c + f_s A_s + f_r A_r \qquad (24)$$

Wherein A_c = net area of concrete section

$= A_g - A_s - A_r.$

A_s = cross-sectional area of longitudinal bar reinforcement.
A_r = cross-sectional area of the steel or cast-iron core.
f_r = permissible unit stress in metal core, not to exceed 16,000 lb. per sq. in. for a steel core; or 10,000 lb. per sq. in. for a cast-iron core.

The remaining notation is that of Section 1103.

(*b*) *Details of Metal Core and Reinforcement*—The cross-sectional area of the metal core shall not exceed 20 per cent of the gross area of the column. If a hollow metal core is used it shall be filled with concrete. The amounts of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shell outside the spiral shall conform to the limiting values specified in Sections 1103 (b), (c) and (d). A clearance of at least 3 inches shall be maintained between the spiral and the metal core at all points except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to 2 inches.

(*c*) *Splices and Connections of Metal Cores*—Metal cores in composite columns shall be accurately milled at splices and positive provision shall be made for alignment of one core above another. At the column base, provision shall be made to transfer the load to the footing at safe unit stresses in accordance with Section 305 (a). The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of load from reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members such as billets, brackets or other positive connections; these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of Formula (24) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, in accordance with Formula (22), all floor loads brought onto the column at levels between the metal brackets or connections. In applying Formula (22), the value of A_g shall be interpreted as the area of the concrete section outside the metal core, and the permissible load on the reinforced concrete section shall be further limited to $0.35\ f'_cA_g$. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders.

(*d*) *Permissible Load on Metal Core Only*

The metal cores of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

1106—Combination Columns

(*a*) *Steel Columns Encased in Concrete*

The permissible load on a structural steel column which is encased in concrete at least 2½ inches thick over all metal (except rivet heads) reinforced as hereinafter specified, shall be computed by Formula (25).

$$P = A_rf'_r\left(1 + \frac{A_g}{100\ A_r}\right) \qquad (25)$$

Wherein A_r = cross-sectional area of steel column.
f'_r = permissible stress for unencased steel column.
A_g = total area of concrete section.

The concrete used shall develop a compressive strength, (f'_c) of at least 2000 lb. per sq. in. at 28 days. The concrete shall be reinforced by (the equivalent of) welded wire mesh having wires of No. 10 W. & M. gage, the wires encircling the column being spaced not more than 4 inches apart and those parallel to the column axis not more than 8 inches apart. This mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface and shall be lap-spliced at least 40 wire diameters and wired at the splice. Special brackets shall be used to receive the entire floor load at each floor level. The steel column shall be designed to carry safely any construction or other loads to be placed upon it prior to its encasement in concrete.

(*b*) *Pipe Columns*

The permissible load on columns consisting of steel pipe filled with concrete shall be determined by Formula (26).

$$P = 0.22\, f'_c A_c + f'_r A_r \qquad (26)$$

The value of f'_r shall be that given by Formula (27).

$$f'_r = \left(18{,}000 - 70\frac{h}{K}\right) F \qquad (27)$$

Wherein f'_r = average unit stress in metal core

h = unsupported length of column

K = least radius of gyration of metal core section

$$F = \frac{\text{yield point of pipe}}{45{,}000}$$

If the yield point of the pipe is not known, the factor F shall be taken as 0.5.

1107—Long Columns

(*a*) The maximum permissible load P' on axially loaded reinforced concrete or composite columns having a length, (h), greater than 10 times the least lateral dimension, (d), shall be given by Formula (28).

$$P^1 = P\left(1.3 - .03\frac{h}{d}\right) \qquad (28)$$

where P is the permissible axial load on a short column as given by Formulas (22) and (24).

The maximum permissible load P' on eccentrically loaded columns in which $\frac{h}{d}$ exceeds 10 shall also be given by Formula (28), in which P is the permissible eccentrically applied load on a short column as determined by the provisions of Sections 1109 and 1110. In long columns subjected to definite bending stresses, as determined in Section 1108, the ratio $\frac{h}{d}$ shall not exceed 20.

1108—Bending Moments in Columns

(*a*) The bending moments in the columns of all reinforced concrete structures shall be determined on the basis of loading conditions and restraint and shall be provided for in the design. When the stiffness and strength of the columns are utilized to reduce moments in beams, girders, or slabs, as in the case of rigid frames, or in other forms of continuous construction wherein column moments are unavoidable, they shall be provided for in the design. In building frames, particular attention shall be

given to cases of unbalanced floor loads on both exterior and interior columns and of eccentric loading due to other causes. Wall columns shall be designed to resist moments produced by

(1) Loads on all floors of the building

(2) Loads on a single exterior bay at two adjacent floor levels, or

(3) Loads on a single exterior bay at one floor level

Resistance to bending moments at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffness and conditions of restraint.

1109—Combined Axial and Bending Stress

(a) In reinforced concrete columns subjected to bending moments, the recognized methods of analysis shall be followed in calculating the stresses due to combined axial load and bending. The maximum fiber stress in compression and (in the case of large eccentricities of loading) the tensile stress in the vertical bars will govern the design. The gross area of both spiral and tied columns shall be used in the computations.[2]

1110—Permissible Combined Compressive and Tensile Stress

(a) The maximum permissible compressive fiber stress, f_c, in eccentrically loaded columns shall be given by Formulas (30) and (31).

For spiral columns

$$f_c = \frac{0.22\, f'_c + f_s p_g}{1 + (n-1)\, p_g} \left[\frac{1 + \frac{ec}{R^2}}{1 + .8\frac{ec}{R^2}}\right] \qquad (30)$$

For tied columns

$$f_c = \frac{0.154\, f'_c + .7\, f_s p_g}{1 + (n-1)\, p_g} \left[\frac{1 + \frac{ec}{R^2}}{1 + .8\frac{ec}{R^2}}\right] \qquad (31)$$

wherein the notation is that of Section 1103 and 1109.

The permissible tensile stress in the longitudinal reinforcement may equal that specified for flexural members, provided however that splices in the tensile steel at or near the section of maximum column moment are capable of developing fully the yield point strength of the reinforcement.

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[2]For preliminary designs it will usually give satisfactory results to compute the combined fiber stress in compression on the basis of an uncracked section of the column, using Formula (29).

$$f_c = \frac{P}{A_g}\,\frac{\left(1 + \frac{ec}{R^2}\right)}{1 + (n-1)\, p_g} \qquad (29)$$

Wherein e = eccentricity of resultant load, measured from the gravity axis.
c = distance from gravity axis to extreme fiber in compression.
R = radius of gyration of equivalent concrete section.
$n = \frac{30{,}000}{f'_c}$

This will result in a fairly accurate design if the eccentricity is less than ½ the over-all column depth and the value of $p_g n$ is 0.3 or more.

The term $\frac{ec}{R^2}$ may be replaced by the value $\frac{6e}{t}$ for rectangular columns and $\frac{8e}{t}$ for round columns without appreciable error (t = over-all depth of section). This design may then be analyzed by more accurate methods to insure that permissible stresses are not exceeded.

1111—Wind Stresses

(*a*) When the allowable unit stress in columns is modified to provide for combined axial load and bending, and the stress due to wind loads is also added, the total shall still come within the allowable values specified for wind loads in Section 603 (*c*).

1112—Monolithic Walls

(*a*) The working stresses in reinforced concrete bearing walls with minimum reinforcement as required by Section 1112 (i), shall be $0.2f'_c$ for walls having a relation of height to thickness of 10 or less, and shall be reduced on the basis of formula (28), to $0.11 f'_c$ for walls having a relation of height to thickness of 25. When the reinforcement in bearing walls is designed, placed and anchored in position as for tied columns, the working stresses shall be on the basis of formulas (28) and (31), as for columns. In the case of concentrated loads, the length of the wall to be considered as effective for each shall not exceed the center to center distance between loads, nor shall it exceed the width of the bearing plus four times the wall thickness. The ratio p_g shall not exceed 0.04.

(*b*) Walls shall be designed for any lateral or other pressure to which they are subjected. Proper provision shall be made for eccentric loads and wind stresses. In such designs the allowable stresses shall be as given in Section 305 (a) and 603 (*c*).

(*c*) Panel and enclosure walls of reinforced concrete shall have a thickness of not less than five inches and not less than one thirtieth the distance between the supporting or enclosing members.

(*d*) Bearing walls of reinforced concrete in buildings of fire-resistive construction shall be not less than six inches in thickness for the uppermost fifteen feet of their height; and for each successive twenty-five feet downward, or fraction thereof, the minimum thickness shall be increased one inch.

(*e*) In buildings of non-fire resistive construction bearing walls of reinforced concrete shall not be less than one and one-third times the thickness required for buildings of fire-resistive construction, except that for dwellings of two stories or less in height the thickness of walls may be the same as specified for buildings of fire-resistive construction.

(*f*) Exterior basement walls, foundation walls, fire walls and party walls shall not be less than eight inches thick.

(*g*) Reinforced concrete bearing walls shall have a thickness of at least one twenty-fifth (1/25) of the unsupported height or width, whichever is the shorter; provided however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of increased thickness.

(*h*) Monolithic walls shall be anchored to the floors, columns, pilasters, buttresses and intersecting walls with reinforcement at least equivalent to ⅜ inch round bars eighteen (18) inches on centers, for each layer of wall reinforcement.

(*i*) Monolithic walls shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall, if of bars, and 0.0018 times the area if of electrically welded wire fabric.[3] The wire of the welded fabric shall be of not less than No. 10 W. & M. gauge. Walls more than eight inches in thickness shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than two inches nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement, shall be placed not less than ¾ inches and not more than one-third the thick-

ness of the wall from the interior surface. Bars, if used, shall not be less than the equivalent of three-eighths inch round bars, nor shall they be spaced more than eighteen inches on centers. Welded wire[3] reinforcement for walls shall be in flat sheet form.

CHAPTER 12—FOOTINGS

1201—Loads

(*a*) Footings resting directly on soil or on piles shall be proportioned as to area or number of piles on the basis of the total column load plus the weight of the footing itself. For computations of moments and shears, an upward reaction per unit area or per pile shall be based on the total column load (not including the weight of the footing itself) divided by the area or by the number of piles.

1202—Sloped or Stepped Footings

(*a*) Footings in which the thickness has been determined by the requirements for shear as specified in Sec. 808, may be sloped or stepped, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

1203—Bending Moment in Footings

(*a*) The critical section for bending moment in a concrete footing which supports a concrete column, pedestal or wall, shall be considered to be at the face of the column, pedestal or wall. For footings under masonry walls, the critical section shall be assumed as halfway between the middle and the edge of the wall. For footings under metallic bases, the critical section shall be assumed as halfway between the face of the column or pedestal and the edge of the metallic base.

(*b*) The bending moment at the critical section in a square footing, or in a rectangular footing having its side not greater than one and one-half times its width, shall be computed from the load on trapezoids bounded by the line of the critical section for moment, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of the trapezoid shall be considered as applied at a distance equal to six-tenths of the projection of the footing from the line of critical section for moment. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity.

(*c*) For a round or octagonal concrete column or pedestal, the face of the column shall be taken as the side of a square of an area equal to the area enclosed within the perimeter of the column or pedestal.

1204—Shearing and Bond Stresses

(*a*) The critical section for diagonal tension in footings on soil shall be assumed as a vertical section at a distance (d) from the face of the column or pedestal supported by the footing.

In footings on piles the critical section shall be assumed at a distance $\frac{d}{2}$ from the face of the column or pedestal, and any piles whose centers are at or within the section shall be excluded in computing shear.

[3]Expanded metal has been omitted until a specification can be formulated.

(*b*) For shearing stresses see Sec. 808.

(*c*) The critical sections for bond shall be assumed at the face of the column or pedestal; at vertical planes where changes occur, and near the end of the bending moment reinforcement.

(*d*) For bond stresses see Sec. 901 to 905.

1205—Transfer of Stress at Base of Column

(*a*) The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by dowels. There shall be at least one dowel for each column bar, and the total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the columns. The dowels shall extend up into the column and down into the pedestal or footing the distance required for lap of longitudinal column bars (see Sec. 1103).

(*b*) The permissible compressive unit stress on top of the pedestal or footing directly under the column shall not be greater than that determined by formula (32).

$$r_a = 0.25\, f'_c \sqrt[3]{\frac{A}{A'}} \quad \text{(32)}$$

(*c*) In sloped or stepped footings, A may be taken as the area of the top horizontal surface of the footing, or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of one vertical to two horizontal.

1206—Pedestals and Footings (Plain Concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on critical sections as determined for reinforced concrete footings.

Recommendations for Placing Concrete by Vibration*

Report of Committee 609, Vibration of Concrete

A. E. LINDAU, CHAIRMAN; H. L. FLODIN, SECRETARY

INTRODUCTION

The adoption of high frequency vibrators for placing concrete has been more rapid than the progress in acquiring basic information on such factors as frequency, amplitude, size of vibrator, type of vibrator, period and method of application and others. In preparing a recommended practice for vibration, therefore, many of the requirements must be based on the well established principles of concrete making. It should be realized that the application of vibration to concrete is somewhat of an art which can not be prescribed in a set of rules; that all of the recommendations given herewith are not applicable to every job and that they are subject to change as knowledge of the subject increases and with developments in the equipment.

TYPE OF VIBRATOR

It appears obvious that vibrators applied directly to the concrete are more efficient than those attached to forms, as in the latter type much of the energy is absorbed by the forms. Also, vibrators that are inserted into the mass of concrete are more effective than those applied to the top surface. Surface vibration of slabs should be used only where internal vibration is impracticable. Such vibration may bring an excess of mortar to the top causing subsequent scaling.

Internal vibrators, therefore, should be used in all sections which are sufficiently large for insertion and manipulation of this type. This includes practically all mass work such as dams and retaining walls and most reinforced sections such as occur in bridges and buildings. Internal vibrators may be supplemented by platform or screed type

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This report, written by the Secretary of the Committee in consultation with the Chairman, was submitted to the members of the committee as appointed late in 1934: I. E. Burks, Miles N. Clair, R. E. Davis, R. B. Gage, C. M. Hathaway, M.I.McCarty, F. H. Jackson, W. R. Johnson, Ben Moreell, A. W. Munsell, D. E. Parsons, O. G. Patch, T. C. Powers, P. C. Peterson, C. W. Pierce, F. V. Reagel, T. E. Stanton, Lewis H. Tuthill, M. O. Withey. The report has been revised in conformity with their suggestions. Discussion and criticism are now generally invited, based on field or laboratory data with a view to a revised report as soon as revised opinion justifies it.

vibrators in the event that satisfactory surfaces can not be obtained with the internal type alone.

Form vibrators should be used where it is impossible to use either the internal or surface type. Form vibrators, therefore, should be used for heavily reinforced thin walls, pipe and other precast products. They should be attached to or held on the forms in such manner as to effectively transmit the vibration to the concrete and so that the principal path of motion of the vibration is in a horizontal plane. In some products manufacture they may be attached to a table on which the form or gang of molds may be set.

There are not now sufficient data to warrant the recommendation of any specific type of vibrator for pavement work. Various types of equipment are available. On some finishing machines, vibrators are mounted on the screeds which are heavier and wider than usual. On some equipment the vibrating member is suspended between the screeds and does not rest upon the forms.

Some excellent results have been secured in tests and trial runs with pavement vibrators. While state highway departments have not adopted vibration for pavements, this is possibly because further improvements in apparatus and methods of application are expected. Indications are that equipment now available can be utilized to advantage in improving durability and strength of pavements or in producing pavements comparable with those produced by the usual methods of finishing but at lower cost.

NUMBER AND CAPACITY OF VIBRATORS

Vibrators should be sturdily built and should be of such capacity and used in such number as to insure vibration throughout the entire volume of concrete at the desired rate of placing. For open form work where concrete of harsh consistency and large aggregate is delivered from large buckets, large diameter vibrators and in some cases heavy 2-man vibrators may be used to advantage, supplemented by smaller vibrators along the forms. In any case, a sufficient number of vibrators of ample capacity should be provided so that the placing operation can keep up with the output of the mixers. Reserve vibrators should be at hand for use when others are being serviced. No job of any appreciable size should be attempted with a single vibrator.

While the work that has been done to determine the inter-relation of amplitude and frequency of vibration and characteristics of the mix has not been comprehensive, sufficient work has been done to indicate the effects of variations in these factors. Withey* shows that

*"The Effects of Frequency of Vibration in Making Concrete Beams," by M. O. Withey, presented at Fifteenth Annual Meeting of the Highway Research Board, Washington, D. C., Dec. 5 and 6, 1935.

the effectiveness of internal vibrators improves by increasing the frequency or the amplitude. In tests made on beams using internal vibrators operated at frequencies from 3600 to 7000 impulses per minute, the time required for effective vibration was greatly reduced by increasing the frequency. Under the conditions of these tests it was estimated that concrete of ½-in. slump required 90 seconds vibration at 4000, 45 seconds at 5000, and 25 seconds at 6000; concrete of no slump required 200 seconds at 4000, 80 seconds at 5000, 50 seconds at 6000, and 40 seconds at 7000 impulses per minute. It appears that for most work, high frequencies, 5000 impulses per minute or more, are desirable. Frequencies referred to are the frequencies when the vibrator is submerged in the concrete. The frequency of under-powered equipment as measured when the vibrator is operating free of load may be greatly reduced when the vibrating element is submerged in concrete.

In general, form vibrators are most effective at a high frequency and small amplitude. Too great an amplitude on form vibrators may have a tendency to cause so much movement of forms as to pump air into the concrete.

The higher the frequency or the greater the amplitude, the more rugged must be the equipment. The development since vibrating equipment was first introduced has been toward higher frequencies and sturdier machines. Vibrators should not be operated at higher frequencies than those for which they are designed as the wear is excessive.

Too much emphasis can not be placed on the necessity for sturdy equipment if full advantage of vibration is to be secured, particularly where large aggregates are used as in dams and similar heavy work.

PLANT LAYOUT AND EQUIPMENT

On many jobs, the character of the mix and rate of placing have been determined by the equipment and layout of the plant rather than by the methods or conditions of placing. The full advantages of vibration can not be realized under these conditions. Consistency and harshness of mix should be limited by the vibrating equipment used and the placing conditions rather than by the mixing and transporting equipment and plant layout.

In the case of pavement concrete, tests and trial runs indicate that the equipment now available is capable of compacting any concrete successfully which can be handled efficiently by the other construction units. Maximum benefits can not be expected until the mixing and distributing equipment now in use have been redesigned so as to

handle efficiently the dry harsh mixes most suitable for vibration in pavements.

Vibrators should not be expected to eliminate or correct segregation. Vertical drop is the solution to this problem, first as applied to concrete dropping into place in the forms, then back through all handling operations to the mixer. Concrete should be confined at all points of change in direction of flow. In all cases flow of concrete into forms, hoppers or buggies should be directed into the fresh concrete and not against the sides. Sloping discharge and twin discharge hoppers should be replaced with hoppers having only one discharge and that arranged in the bottom so that the concrete will drop vertically into buggies, cars or belts receiving it. Equipment filling such hoppers should be arranged so that concrete will fall into them vertically without separating. For handling expediently the relatively drier concretes which become practical with vibration, extra large hopper gates and throats to chutes and articulated down pipes are necessary. Twice the usual area is a safe precaution.

FORMS

Forms should be rigid enough to withstand the pressure caused by the head of concrete made plastic by the vibrating action. In sections where concrete may be placed rapidly as in columns and thin walls, the concrete throughout the full height of the section may be made plastic and the pressure would be equivalent to that of a fluid having the weight of concrete. Thus in reinforced concrete work such as bridges and buildings, the pressure is noticeably increased and for this work an increase of approximately twenty-five per cent in the number of studs and walers over those used for hand-placing should be made. Some contractors have used such rigid forms for hand-placed concrete that they do not find it necessary to make this increase. The thickness of sheathing need not be changed if studs and walers are ample. In heavy mass work little or no change is required in forms.

When forms are vibrated they should be rigid enough to prevent pumping of air into the concrete.

Forms should be watertight. There is a greater tendency for loss of mortar through leaky forms when vibration is used than with hand methods. Such leaks cause unsightly sand streaks and in some cases leave the coarse aggregate exposed.

When internal vibrators are used, care should be taken to prevent scarring or roughening of forms by operating vibrators against them.

ADJUSTMENT OF MIX

A series of systematic trial batches should be placed from which a mix should be selected having cement and water contents which will

meet the specifications and which becomes plastic and compacted under the conditions of placement. From the standpoint of economy, the mix should be such that the vibrators can be operated near their full capacity when concrete is placed at the desired rate.

The following method of selecting the most suitable mix from trial batches has been suggested. A mix is chosen which is estimated from previous experience or preliminary tests to require a water-cement ratio slightly below that specified and which has greater plasticity than required by the placing conditions. Placing is then started, using the minimum amount of water which will permit placing at the required rate. From the weights of materials used in the batch including the amount of water added and that introduced as moisture in the aggregates, the total quantity of water per cubic yard of concrete is computed. In successive batches small reductions in the amount of sand with compensating increases in the amount of coarse aggregate are made, adjusting the consistency of each mix by varying the amount of water until determination is made of the minimum amount of water which will permit the vibrators to keep up with production. This procedure is continued until no further reductions in water content are made possible by reductions in sand. This mix will appear harsh and additional water will be required to permit placing at the required rate. The need for additional water is evidence that less than the proper quantity of sand is being used. The next step, therefore, is to hold the amount of water constant and increase the amount of sand up the the maximum which can be used with this minimum amount of water.

It will probably be found that the water-cement ratio of this mix is not the same as that specified. Compensation for this may be made by the rule of constant water content stated by Slater and Lyse.* In a given series of mixes of the same consistency and aggregate combination, the total amount of water per cubic yard of concrete remains substantially constant regardless of the variation in cement content.

In the series of trial batches, the minimum amount of water per cubic yard of concrete has been determined. Dividing this by the specified water-cement ratio gives the cement content which can be used. If the actual cement content in the final mix is other than this, the trial water-cement ratio is wrong and the cement content must be changed to give the correct value. For example, if it is found that 30 gal. of water per cubic yard of concrete can be used and the specified water-cement ratio is 6 gal. per sack, the cement content should be 5 sacks per cubic yard. If the actual cement content in the final mix of the

*Journal, Am. Concrete Inst., June, 1930, Vol. 26, p. 831.

trial batches is 5½ sacks, it is necessary to reduce the cement by ½ sack. Compensation for this reduction in cement is made by adding the equivalent absolute volume of sand.

Similar trials may be made with other materials to determine the best gradation of aggregates or combination of the available materials.

On pavement construction the selection of the most suitable mix is relatively simple. Starting with the mix used for ordinary finishing, sand and gravel may be added to this base mix, keeping the water-cement ratio constant, until the slump has been reduced to one inch and the percentage of sand in terms of total aggregates has been reduced approximately 5 per cent. For example, if 35 per cent of sand is used in the base mix, it may ordinarily be reduced to about 30 per cent. If it is desired to maintain the same cement content as in the base mix, the quantity of water should be reduced and the ratio of sand to total aggregate changed as above until the slump has been reduced to 1 inch, keeping the total weight of aggregate per sack of cement constant.

PROCEDURE IN VIBRATING

The procedure to follow in vibrating should vary with the conditions of placing, type of vibrators used and character of mix. In so far as possible, concrete should not be transported in the forms by causing it to flow laterally an appreciable distance from one point to another. It should be put where it is to stay with the next batch beside it and so on along the work until a shallow lift is completed. The vibration should not be dissipated in lateral motion but should be concentrated in vertical settlement and consolidation of the concrete.

Internal vibrators should be applied at distances not greater than the radius through which the vibration is visibly effective. As a rule the effects of vibrators can be seen for a distance of 3 ft. or more from the machine, but they should be applied at closer intervals than this. Generally the intervals between points of insertion should not be less than about two feet. In any case the vibrators should be inserted at sufficiently short intervals that the vibrated areas overlap without omission of any area. The vibrators should be inserted and withdrawn slowly and should be operated continuously while being withdrawn. Vibrators should be inserted to a sufficient depth to vibrate the bottom of each layer effectively but should not be allowed to penetrate partially hardened concrete which will not become plastic under the vibratory action. Vibrators should not be applied directly to steel which extends into partially hardened concrete. They should be applied close enough to the forms to vibrate the surface concrete effectively.

Vibrating equipment for compacting concrete in slabs should compact the slab to its full depth and the concrete should be spread uniformly in front of it.

Vibration should be such that the concrete becomes uniformly plastic and there should be at least 15 seconds of vibration per square foot of top surface of each layer, computed on the basis of the visibly affected radius and taking overlapping into consideration.

Vibration should not be continued in any one spot to the extent that pools of grout are formed. No more than the slight excess of mortar necessary to indicate that the voids are completely filled should be brought to the surface. Concrete of wet consistency should be vibrated sparingly to avoid separation of the materials. It is considered bad practice to vibrate concrete having slump more than 5 inches and it appears entirely practicable to use an upper limit of 4 inches for ordinary building construction.

When concrete is deposited from buckets, vibration should be started at the point of deposit by internal vibrators held in an inclined position and inserted and withdrawn at the proper intervals until the batch is level. Any separation of coarse aggregate from the mortar should be corrected by shovelling the aggregate into the mortar and vibrating it back into the mass.

With harsh-working, non-plastic mixtures, air bubbles along form surfaces should be reduced to a minimum by vigorous, systemized vibration as close to the forms as possible without damaging them. This should provide a flow of mortar along the forms. If there is a tendency for the formation of water pockets it may be necessary to keep the vibrator farther away from the forms. With plastic mixtures, light vibration may be supplemented by spading along forms to reduce air bubbles to the minimum.

Honeycombed areas are an indication of insufficient mortar, insufficient vibration or poor distribution of vibration. They should be avoided by providing enough mortar for the conditions of placing and vibrating.

Vibration may cause "bleeding" or separation of free water from the mix at a more rapid rate than would occur in the same mix placed by hand. Indications are that bleeding is due mainly to the physical properties of the mix and the characteristics of the materials. It should be prevented by reducing the amount of water or by increasing the percentage of fine particles passing the 50 and 100-mesh sieves. While less mortar usually can be used in vibrated concrete than is required in hand-placed concrete, the requirement for sufficient fine particles in the mix to hold the water must still be met.

In vibrating and finishing top surfaces which are exposed to weathering or wear, considerable care should be taken to avoid drawing water or laitance to the surface. In relatively high lifts, the top layer should be rather shallow and placed as stiff as possible with sufficient vibration to compact and finish the concrete.

GENERAL DISCUSSION

Effects of Vibration

The most important effect of vibrating fresh concrete is to make the mixture more plastic. Density may be slightly increased due to reduction in the amount of entrapped air. Internal vibration improves the bond between successive layers of concrete.

The water-cement ratio-strength relation for hand-placed concrete holds also for vibrated concrete. The vibration simply extends this relationship into the area of stiff mixes which are impracticable to place by hand. There is, therefore, a choice of either improving the quality of concrete (strength, watertightness, durability, etc.) without increasing the cement factor or securing the same quality as hand-placed concrete with a more economical mixture. The first of these is brought about by reducing the water-cement ratio; the second is brought about by increasing the amount of aggregate used with a given water-cement ratio.

Reduction in Slump

How stiff a mix can be placed will depend on many factors such as the effectiveness of the vibrators, character of mix and placing condition. Comess [25] reports the use of ¾-in. slump with 2-in. aggregate in large, open form work.

In reinforced concrete structures a greater range of consistencies has been used. Hathaway [17] states that in bridge construction, where 4- to 6-in. slump is required without vibrators, 2½- to 3½-in. slump can be used with small vibrators, and 1- to 2-in. slump with large vibrators. Brett [12] reports using 2-in. slump in bridges, Stanton [21] reports a range of 1- to 4-in. slump for the various sections of the San Francisco-Oakland Bay Bridge. Tuthill [22] states that where 8-in. slump had been used it was reduced to 5-in. with vibration and for slab and beam floors the slump was reduced from 6-in. to 3-in. Benkelman [10] reports vibrating 1½-in. slump concrete in precast reinforced concrete piers. Shenk [13] reports 2-in. slump concrete for thin precast roof slabs. Jackson [23] reports that in pavement work a slump of from ¾-in. to 1-in. seems to be about the driest consistency which can be handled by the various construction units now in use without having segregation on the subgrade.

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[25] This and subsequent references are to appended bibliography.

Reduction in Sand Factor

While the proportion of sand can be reduced with vibration, it is necessary to exercise precaution against undersanding as it is attended by more difficulties in handling and in getting proper results than is warranted by the small economies or increases in strength or density which are made by so doing. Tuthill[22] says that the fine material smaller than ⅜-in. should compose not less than 33 to 40 per cent of the mixed aggregate, depending on the shape of the particles, the maximum size of aggregate and cement content. Powers[5] found that the sand factor could be reduced from 39 to 31 per cent in mixes used in dams. Johnson[19] says that the sand factor in a mix requiring 35 to 37 per cent for hand placing could be reduced to 31 to 33 per cent when the concrete was vibrated.

Jackson[23] states that the bulk of experience in vibrating pavements indicates that the percentage of sand in terms of the weight of total aggregate may be about 5 per cent less when vibration is used, that is, mixtures which for ordinary placing would be designed for 35 per cent sand should, with similar materials, be proportioned on the basis of about 30 per cent sand.

The lower limit in sand factor has been exceeded when coarse material, which will not be absorbed by the mixture under vibratory action, lies loosely on top of the mass. For best results under average conditions the sand factor should be somewhat above the low limit.

Test for Effectiveness of Vibrator

It has been suggested that for ordinary service in structural or mass concrete, vibrators must be sufficiently powerful to affect the concrete visibly over a radius of at least 18 in. assuming a properly designed mix with less than 1-in. slump.

Comparisons between different vibrators are best made by actually using them simultaneously. By careful observation when they are operating under similar conditions the most effective can be selected.

Judging When Vibration is Sufficient

The ability to judge when concrete has had sufficient vibration can be developed only by experience. One of the evidences of sufficient vibration is a visible line of cement paste at the junction of the concrete and the forms or between the concrete and the surface of the steel reinforcement. Another evidence is a level top surface with just enough mortar for finishing.

Capacity of Vibrators

The volume of concrete that can be handled per vibrator varies widely on account of the great number of variables on different jobs.

On the San Francisco-Oakland Bay Bridge, 2-man vibrators of high frequency handled from 25 to 40 cu. yd. of mass concrete. Johnson[19] states that in general the quantity will vary from 12 to 36 cu. yd. per hour. Hathaway [17] reports that on such construction as bridges, the smaller vibrators can handle from 5 to 6 cu. yd. per hour, whereas the sturdier, well-powered vibrators have handled from 15 to 20 cu. yd. per hour. Tuthill [22] suggests that properly powered and operated internal vibrators will take care of 10 to 20 cu. yd. of concrete per hour depending upon its accessibility and how well it is placed.

In general, a larger volume of concrete having aggregate of gravel and fine sand can be handled, within a limited time, than a harsher mix containing crushed rock and coarse sand.

SELECTED BIBLIOGRAPHY ON VIBRATION OF CONCRETE

(1) Concreting Methods at Chute á Caron Dam, by I. E. Burks. JOURNAL, Am. Concrete Inst., February, 1930, *Proceedings*, Vol. 26, p. 315. Platform vibrators permitted a stiffer mix than could be placed in same time by hand.

(2) The Effect of Vibration on the Pressure of Concrete Against Formwork, by L. W. Teller. *Public Roads*, March, 1931, p. 11. Tests of 12-ft. columns indicated pressure equivalent to fluid pressure of full column of concrete made plastic by vibration. With hand placing, pressure increased hydrostatically up to certain value, after which it decreased.

(3) Concreting the Calderwood Tunnel, by W. R. Johnson. JOURNAL, Am. Concrete Inst., June, 1931, *Proceedings*, Vol. 27, p. 1189. Electric tampers clamped to forms were run intermittently during placing operations and were, no doubt, responsible to some extent for smooth surface obtained.

(4) Compaction of Concrete Through the Use of Vibratory Tampers, by R. E. Davis and H. E. Davis. JOURNAL, Am. Concrete Inst., June, 1933, *Proceedings*, Vol. 29, p. 365. Pressure of concrete in agitation is substantially same as that produced by a liquid having same density as concrete. After cessation of vibration, pressure drops off rapidly and in few minutes is same as hand tamped concrete. Compaction is secured more rapidly with vibration or in a given time, concrete of drier consistency can be placed. Under one condition investigated, time of placing by vibration was 1/3 time required for hand-placing or concrete of 1-in. slump could be placed by vibration in same time as 6½-in. slump concrete by hand. Internal vibrators, properly manipulated, eliminate segregation and improve bond between successive layers.

(5) Vibrated Concrete, by T. C. Powers. JOURNAL, Am. Concrete Inst., June, 1933, *Proceedings*, Vol. 29, p. 373. Benefits derived from vibration result from ability to handle concrete of lower water content. Field studies of mass concrete indicated it was possible to reduce water one gallon per sack of cement on project investigated. Tests show that quality of cement paste is chief factor determining strength for both vibrated and hand placed concrete. Freezing and thawing tests indicate no impairment in resistance to weathering, due to vibration. Full benefit of vibration is obtained by use of dry mixes having minimum of sand. Gives example of increasing strength

of 5-sack concrete 1350 p. s. i. by using drier mix and an additional 900 p. s. i. by reducing sand to practical minimum.

(6) The Use of Vibration in the Manufacture of Concrete Products, by Miles N. Clair. JOURNAL, Am. Concrete Inst., June, 1933, *Proceedings*, Vol. 29, p. 383. Discusses relation of frequency, amplitude and period of vibration to character of mix and gives practical values for various types of precast products.

(7) Vibratory Finishing Machine for Concrete Pavements, by F. V. Reagel. JOURNAL, Am. Concrete Inst., June, 1933, *Proceedings*, Vol. 29, p. 391. Tests by Missouri Highway Department indicate possibility of reducing unit cost of concrete in pavements without sacrifice in quality. Vibrated mixtures gave 12 per cent higher strength than unvibrated mixtures of same cement content. This increase was due to reduction in water content.

(8) High Frequency Vibratory Machines for Concrete Placement, by M. I. McCarthy. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 49. Discusses types of vibrators and work for which each is adapted.

(9) Placement of Concrete by Mechanical Vibration, by A. W. Munsell. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 54. Field observations of frequency, amplitude and period of vibration in relation to placing conditions.

(10) Vibration on Michigan Bridge Work, by A. C. Benkelman. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 57. Experiences with three types of vibrators indicate satisfactory results.

(11) Vibrating Equipment in a Cast Stone Plant, by G. B. Pickop. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 59. Due to adoption of vibration, strengths average about 100 per cent higher than formerly, absorption has been reduced from 7 to 3 per cent, products can be polished in 3 days instead of 7 to 14 days and a wider range of textures is possible.

(12) Fabricating 36-in. Reinforced Concrete-Steel Cylinder Water Mains, by J. F. Brett. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 61. Vibrators were attached to forms for casting 60,000 ft. of concrete water pipe. Each pipe was 16½ ft. long and weighed 5 tons.

(13) Vibration in Making Roof Deck Slabs, by A. B. Shenk. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1933, *Proceedings*, Vol. 30, p. 63. Greater amplitude causes concrete to flow more rapidly. Higher speeds seem to produce slower movement of concrete into place but with corresponding increase in density of finished product.

(14) Effect of Vibration and Delayed Finishing on the Quality of Pavement Slabs, by F. H. Jackson and W. F. Kellerman. *Public Roads*, Oct., 1933, p. 129. Slump could be reduced from the required 2½-in. for hand finishing to 1-in. when finished by vibration. Average flexural strength was greater in vibrated sections of 1-in. slump concrete than in standard-finished concrete and also greater where coarse aggregate was increased by one-fourth part. Vibration did not adversely affect hardness of surface.

(15) Vibrating Concrete at Pine Canyon Dam, by S. B. Morris. JOURNAL, Am. Concrete Inst., March-April, 1934, *Proceedings*, Vol. 30, p. 305. Use of vibrators gave higher strengths due to stiffer mixtures used. On upstream face of dam not a single honeycomb or "bug-hole" was found and not over

half a dozen such places on the flat downstream face. Stepping up speed from 2900 r. p. m. to 4700 r. p. m. improved results.

(16) Bonding of New Concrete to Old at Horizontal Construction Joints, by R. E. Davis and H. E. Davis. JOURNAL, Am. Concrete Inst., May-June, 1934, *Proceedings*, Vol. 30, p. 422. Vibration improves bond between layers of concrete.

(17) Practical Application of Vibration, by C. M. Hathaway. JOURNAL, Am. Concrete Inst., March-April, 1935, *Proceedings*, Vol. 31, p. 420. Illinois Highway Department requires vibration in construction of bridges. Slumps of concrete to be vibrated range from 2½ to 3½ in. with small vibrators and from 1 to 2 in. with large vibrators, slump of concrete not vibrated is 4 to 6 in. for this type of work. Air bubbles have been largely eliminated by oiling forms and by supplementary hand spading.

(18) Vibrated Concrete in Pavement Slabs, by F. V. Reagel. JOURNAL, Am. Concrete Inst., March-April, 1935, *Proceedings*, Vol. 31, p. 424. Difficulties with equipment tested are discussed. Greater control is necessary to avoid sudden and extreme variations in aggregate moisture content. Multiple sized aggregate is recommended.

(19) Vibration of Concrete, by W. R. Johnson. JOURNAL, Am. Concrete Inst., March-April, 1935, *Proceedings*, Vol. 31, p. 429. Recommendations for vibrating concrete are made based on field and laboratory experience.

(20) Freezing and Thawing, Permeability and Strength Tests on Vibrated Concrete Cylinders of Low Cement Content, by M. O. Withey. JOURNAL, Am. Concrete Inst., May-June, 1935, *Proceedings*, Vol. 31, p. 528. Tests indicate that extremely lean mixtures give good resistance against freezing and thawing, high strengths and watertightness when concrete is compacted by vibration. Better consolidation can be secured in shorter periods by using higher frequencies.

(21) Vibration of Concrete on San Francisco-Oakland Bay Bridge, by T. E. Stanton, Jr. JOURNAL, Am. Concrete Inst., May-June, 1935, *Proceedings*, Vol. 31, p. 539. Various types of vibrators used with good results. Vibrators used on pavements, one of which was made with lightweight aggregates.

(22) Vibration as an Aid in Placing Better Concrete, by L. H. Tuthill. JOURNAL, Am. Concrete Inst., May-June, 1935, *Proceedings*, Vol. 31, p. 545. Emphasizes importance of proper handling and placing of concrete to secure full benefit of vibration. Efficiency of vibrators must not be impaired by using them to transport or remix concrete in forms. Discusses other requirements.

(23. High Frequency Vibration as Applied to the Construction of Concrete Pavements, by F. H. Jackson. JOURNAL, Am. Concrete Inst., May-June, 1935, *Proceedings*, Vol. 31, p. 551. Summarizes experiences of state highway departments and Bureau of Public Roads. Concludes that, if properly applied, vibration can be used to materially improve quality of paving concrete but if maximum benefits are to be secured, present methods of handling concrete must be revised.

(24) Concrete Vibrating Practices in France, by B. Moreell. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1935, *Proceedings*, Vol. 32, p. 66. In addition to American types of vibration, internal "floating" vibrators are used.

(25) Practical Application of Vibration for Placing Concrete, by S. Comess. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1935, *Proceedings*, Vol. 32, p. 68. Vibration treatment required depends on characteristics of mix. Vibration does not cause "bleeding" but causes its earlier appearance. Light vibrators proved ineffective in mass concrete. Greatest dispersion and liberating of air bubbles from face was secured when vibrators were held as closely as possible to form.

(26) Observation on the Use of Vibration in the Field, by T. C. Powers. JOURNAL, Am. Concrete Inst., Sept.-Oct., 1935, *Proceedings*, Vol. 32, p. 74. When unsatisfactory results were secured it was usually due to poor management of crew, improper transporting of concrete, unwise selection of vibrators, operating vibrators below normal speed, improperly designed mixes. Outlines method of designing mixes.

For such discussion of this report as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by July 1, 1936.*

A Study of the Economics of High Strength Concrete in Building Construction*

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INTRODUCTION

During the last few years, the subject of high-strength concrete has been discussed by a number of engineering writers, including A. R. Lord, T. T. Towles, Inge Lyse, H. J. Gilkey and G. C. Ernst.‡ All have pointed out certain advantages and economies to be attained through the use of high-strength concrete. It is the purpose of his paper to analyze savings in building costs due to the use of high grade concrete, and also to indicate some of the less tangible benefits which may be secured.

Consideration of a simple member, such as a plain concrete pier, indicates a great possibility of saving in cost, since if the concrete strength is doubled, the volume of the pier may be halved. However, plain concrete finds little use in buildings, and with the addition of reinforcing steel, the cost is greatly increased. Even in reinforced concrete, the simple compression member still offers the greatest opportunity for the use of high-strength concrete, for when flexure is encountered the use of improved concrete generally requires in turn an increase in the reinforcing steel.

Comparison in general is difficult because of the many factors involved, such as the arbitrary requirements of building codes, labor union rules, wide variations in the cost of materials and in freight and hauling charges, and a great spread in relative costs between cement, aggregates and steel.

The following comparisons will be based on a consideration of the actual strength and cost of various concrete mixtures. It does not

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936.

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‡A. R. Lord, "Design and Cost Data for the 1928 Joint Standard Building Code," *Proceedings*, Am. Concrete Inst., Vol. 25, 1928, p. 537-744.

T. T. Towles, "Advantages in the Use of High-Strength Concretes," Journal, Am. Concrete Inst., May, 1932, *Proceedings*, Vol. 28, p. 607-612.

Inge Lyse, "Relation between Quality and Economy of Concrete," Journal, Am. Concrete Inst., March-April, 1933, *Proceedings*, Vol. 29, p. 325-343.

H. J. Gilkey and G. C. Ernst, Report of Project Committee on the Use of High Elastic Limit Steel as Reinforcement for Concrete, *Proceedings*, Highway Research Board, Vol. 14, Part 1, p. 255-302, 1935.

cover fully the common condition under which material is often wasted today; namely, the practice of using a concrete of much better quality than that utilized in the design computations. This is a matter for cooperation between designer and builder, for there is no object in using an unnecessarily expensive concrete if, for example, the strength of the structure is governed by the design of the reinforcing steel.

MATERIAL COSTS

For the quantitative study of building costs, a set of average costs has been selected which is based on published records from various cities in the United States. This information, both as to the separate materials and as to ready-mixed concrete, together with specific data on cost of materials, handling and placing, have resulted in the following unit values:

Concrete Strength	Price per cu. yd.
2000	$7.20
3000	7.50
4000	7.90
5000	8.40
6000	9.05

Concrete, in place in structure, but not including forms or protection. These costs are based upon the assumption that the concrete will be handled in lots of 100 cu. yds. or more. Similar cost values for reinforcing steel, including fabrication and setting in forms, are as follows:

Bar Steel for slabs, beams, columns, etc.	$ 78 per ton
Stirrups and hot rolled spiral reinforcement	90 per ton
Cold drawn wire spiral reinforcement	100 per ton

It is recognized that these values may all be rather low since they contain no allowance for waste or for various overhead charges, but the whole study is intended to apply particularly to the large, well-organized job, in which the opportunities for saving in cost may be important. The ratio of costs of concrete and steel are believed to be fairly representative. The cost of forms has not been included in this study, since the advantage in form costs in favor of the stronger concrete is of relatively minor importance. Similarly, the costs of curing, protection, and many other incidental charges have not been included here, since they are unaffected by the grade of concrete used.

STUDY OF BUILDING COSTS

The computations to follow are intended to apply to a reinforced concrete building of five to eight stories. While an actual building frame of definite size has not been used, the relative importance of the various elements of the building will be recognized. In choosing a range of concrete strengths for this study the upper limit has been set somewhat high, though it can be attained with mixtures in general use,

while the lower limit may result in concrete of doubtful durability. The range from 2000 to 6000 p. s. i. may be defended on the ground that a consideration of rather extreme limits is needed to indicate clearly the effect of smaller variations.

In the comparisons to follow, the designs will be made in accordance with the 1928 Joint Standard Building Code of the American Concrete Institute and the Concrete Reinforcing Steel Institute, unless otherwise noted. Except for the case of columns, the comparisons made here would not be changed appreciably had the 1936 revision of the code been used instead of the 1928 edition.

FLAT SLABS

Comparative studies have been made of the cost of an interior panel of a flat slab floor, assuming concrete strengths of 2000, 4000 and 6000 p. s. i.; panels 16, 20 and 24 ft. square, and live loads of 100 and 300 p. s. f. The thickness of a flat slab with drop panel is governed by two limitations; a minimum limit of $\frac{1}{32}$ of span (for 2000-lb. concrete) and a strength formula. It is found that the former governs for ordinary spans when the live load is less than 150-170 p. s. f. and the other applies above this limit. This dual specification produces a difference in the basic design for the two live-loads used in this study. Another feature of the code is the provision for varying slab thicknesses inversely with the cube root of the strength of the concrete used. This rule was evidently intended to maintain a constant deflection of slab, and results in an excess of compressive strength when high-strength concrete is employed. On the basis of strength alone, high strength concrete is penalized to some extent in this flat slab code.

The effects of high strength concrete in effecting a saving in amount of concrete, an increase in amount of reinforcing steel, and a saving in total load to be carried by columns and footings, together with the relative total slab costs, are summarized in Table 1. It is evident that the replacement of 2000-lb. concrete with 4000-lb. material, while it reduces the slab thickness 20 to 23 per cent, also increases the amount of reinforcement a somewhat similar amount, and produces only 0 to 4.5 per cent saving in cost. Similarly the use of 6000-lb. concrete reduces the amount of concrete about one-third, increases the steel tonnage 27 to 49 per cent and increases the total cost from 0 to 9 per cent. A further advantage which cannot be applied to the slab costs, is the saving in load transmitted to the substructure, varying from 5 to 16 per cent for the various cases shown. For codes which permit a reduction in column live loads in lower stories, the saving is still greater than that shown.

TABLE 1—FLAT SLAB COST DATA

Based on 1928 A. C. I. Building Code

Span of Square Panel ft.	Live Load lb. per square ft.	Concrete Strength f'_c	Slab Thickness* in.	Per Cent Saving in Amount of Concrete	Per Cent Increase in Amount of Steel	Relative Costs	Per Cent Saving in Load on Columns
16	100	2000	6.0	——	——	100.0	——
		4000	4.75	20.7	21.0	98.2	9.2
		6000	4.25	30.7	36.5	102.4	13.6
	300	2000	7.5	——	——	100.0	——
		4000	5.75	22.5	28.0	100.4	5.3
		6000	5.0	33.2	49.0	109.0	7.7
20	100	2000	7.5	——	——	100.0	——
		4000	6.0	20.7	17.5	96.5	9.8
		6000	5.25	30.7	30.5	101.3	14.7
	300	2000	9.0	——	——	100.0	——
		4000	7.0	22.2	25.0	99.7	6.0
		6000	6.0	33.3	45.0	105.1	9.0
24	100	2000	9.0	——	——	100.0	——
		4000	7.25	20.7	16.0	95.5	11.0
		6000	6.25	30.7	27.5	99.7	16.3
	300	2000	11.0	——	——	100.0	——
		4000	8.5	23.0	24.5	100.7	8.9
		6000	7.25	34.6	42.5	106.0	10.7

*Minimum allowable thickness = $\frac{1}{32}$ of span x $\sqrt[3]{\frac{2000}{f'_c}}$ for all live loads below 150 lb. per sq. ft.

Slab thicknesses shown to nearest quarter inch, but theoretical values used in cost computations.

RECTANGULAR BEAMS AND ONE-WAY SLABS

The rectangular beam is not particularly important in buildings but the one-way slab is very commonly used. However, cost studies of both types of member have been made. Upturned spandrel beams and isolated beams are often designed as rectangular beams. Table 2 presents a study of costs for such beams, under one particular set of conditions. Two extreme limits of design are considered; (1) width of beam held constant, depth varied with change in concrete strength, and (2) depth of beam held constant and width varied. The usual design, in which both depth and width would be varied, lies between these extremes.

In preparing Table 2, the code values of f_s = 20,000 p. s. i., f_c = $.4f'_c$, and nf'_c = 30,000, were used. In part I, with constant depth of beam and balanced reinforcement, it is seen that as higher strengths of concrete are used, the amount of reinforcement increases faster than the volume of concrete decreases; as a result, the cost shows little change, but would probably increase considerably for values of f'_c above 6000 p. s. i. The reduction in total load due to the use of stronger concretes is small. It has been assumed in the foregoing that flexural stress governs the beam design. The shearing stresses become a smaller proportion of the compressive strength as the latter is increased,

TABLE 2—COST DATA FOR RECTANGULAR BEAMS

I. Width of Beam Constant, Depth Varying. Constant Live Load 2400 lb. per lin. ft.

Concrete Strength	Effective Depth in.	Per Cent Reduction in Amount of Concrete	Per Cent Increase in Amount of Steel	Relative Costs	Estimated Per Cent Reduction in Total Load (L. L. + D. L.)
2000	22.0	——	——	100.0	——
3000	18.0	16.7	20.5	97.5	1.8
4000	15.6	26.5	37.5	99.1	3.0
5000	13.9	33.7	52.0	101.2	3.7
6000	12.7	38.7	65.0	106.2	4.3

II. Depth of Beam Constant, Width Varying. Constant Live Load 2400 lb. per lin. ft.

Concrete Strength	Effective Depth in.	Per Cent Reduction in Amount of Concrete	Per Cent Increase in Amount of Steel	Relative Costs	Estimated Per Cent Reduction in Total Load (L. L. + D. L.)
2000	22.0	——	——	100.0	——
3000	22.0	36.0	−3.7	80.5	3.7
4000	22.0	53.0	−5.6	70.5	5.6
5000	22.0	62.5	−6.7	64.5	6.7
6000	22.0	69.1	−7.4	61.5	7.4

Note: Full theoretical reduction of width not always possible. Design may be modified slightly by bond and shear requirements.

TABLE 3—COST DATA FOR ONE-WAY SLABS

Simple Span—Live Load 300 lb. per sq. ft. 12 ft. Span

Concrete Strength	Effective Depth in.	Per Cent Reduction in Amount of Concrete	Per Cent Increase in Amount of Steel	Relative Costs	Estimated Per Cent Reduction in Total Load
2000	7.50	——	——	100.0	——
3000	6.00	17.5	19.0	96.7	4.5
4000	5.10	28.1	36.0	96.3	7.3
5000	4.50	35.2	51.0	99.5	9.0
6000	4.10	40.0	63.5	103.7	10.4

and where stirrups are required the volume of stirrup steel remains constant as the depth of beam is varied. Bond stresses offer no difficulty with high strength concretes, as the induced stresses do not increase as rapidly as do the allowable values.

Referring to part II, of Table 2, very considerable savings may be produced with the higher strengths of concrete by holding the beam depth constant and decreasing the width. Obviously, there would be practical limitations to this, since beam widths are often governed by the spacing of the longitudinal steel. However, there is a large saving in concrete, a small one in longitudinal steel, there is no increase in cross-section due to increased shearing stress, no increase in stirrup steel and the permissible increase in bond stress will help very considerably in reducing the number of reinforcing bars as the width of the beam is decreased.

While actual modifications of beam designs in the utilization of stronger concretes may lie between the two cases illustrated, the second method points the way toward a very considerable economy in costs,

and appears to be the limiting case which practical designs might well approach.

Table 3 gives cost data for a typical one-way slab design. It is evident that the one-way slab falls under case I of Table 2, the rectangular beam of constant width and varying depth. The slab design differs from beam design in one respect; the ratio of dead load to live load on the slab is much larger than it is for the beam. Hence, it is possible with strong concretes to effect a greater reduction in total load for slabs than for beams. The reduction in cost of slabs, as shown in Table 3, is small, but taken together with the reduction in total load to be transmitted to the columns and footings, it may result in a worthwhile saving on a large structure. The cost relations for the one-way slab made with different grades of concrete are seen to be similar to those for flat slabs.

T-BEAMS

The great majority of beams and girders in buildings are designed as T-beams. The T-beam offers less chance for saving through the use of high strength concrete than the rectangular beam because the depth is rarely governed by the concrete fiber stress. Usually there is so much adjacent floor slab to be utilized as a T-flange that the concrete compressive stress is low. The possibility of saving in T-beams lies in a reduction in the width or depth of the stem or web of the beam. Since this is governed by diagonal tensile stresses and by bar spacing requirements, it appears that considerable savings may be effected through reductions in width, in the direction shown in part II of Table 2. The saving, however, is confined to the stem of the T-beam and may be perhaps ⅔ as great as that obtained with the rectangular beam.

COLUMNS

The greatest reduction in cost in a building structure may be secured through the design of the columns. The column is essentially a compression member. The volume of a plain column varies inversely as the strength of the concrete, but such a column is not used. With the addition of compression steel, and of ties or spirals, the savings due to strong concretes are lessened, but still very considerable in amount. Fig. 1(a) shows the relative cost of spirally reinforced columns of different concrete strengths and steel ratios. Following the 1928 A. C. I. code, these columns have spiral reinforcement equal to one-fourth the longitudinal reinforcement. The figure shows that the column cost may be decreased by the use of a low steel percentage and of high concrete strength. However, using low steel percentage alone results in relatively large columns; in addition, from considerations of shrink-

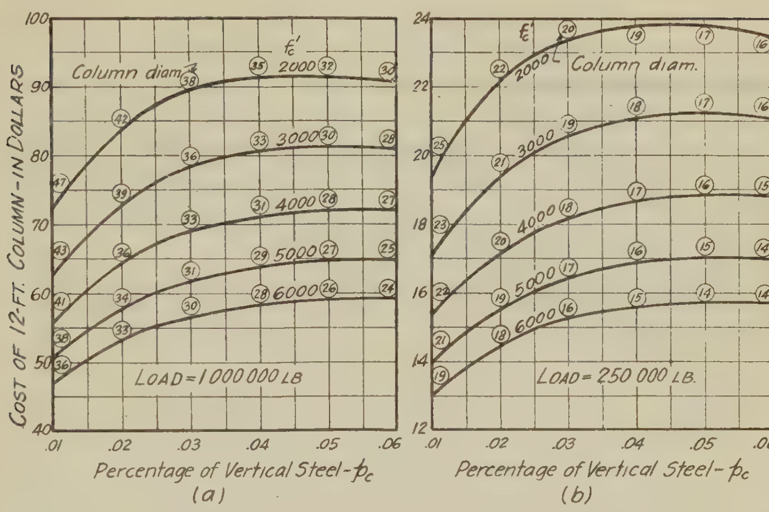

FIG. 1—COST OF SPIRAL COLUMNS UNDER 1928 CODE: (a) LOAD, 1,000,000 LB.; (b) LOAD, 250,000 LB.

age and flow, the minimum percentage of steel is known to produce undesirably high steel stresses with sustained loading. Use of the maximum percentage of steel, while structurally desirable, results in high cost. Some intermediate value, say 2 to 3 per cent, would seem to be a happy medium. If this percentage is combined with a strong concrete, it is evident that both the cost and the column diameter are greatly decreased. This bears out the statement, made nearly thirty years ago by M. O. Withey,* that "cement is an economical reinforcement" for columns. Fig. 1(b) shows a similar study for columns carrying a relatively low load. Fig. 2 shows a comparison of costs of tied and spirally reinforced columns under a medium load. In this figure the steel percentage is based on core areas for spiral columns and on gross area for tied columns. This upsets a direct comparison, but it is evident that nearly all of the tied columns are cheaper than the corresponding spirally reinforced columns. This is caused in part by a lack of balance between the formulas for tied and spiral columns under the 1928 code.

Fig. 3 shows a similar comparison of tied and spiral column costs, based on the column formulas given in the 1936 Proposed Building Regulation for Reinforced Concrete, prepared by Committee 501 of the Institute.† For comparison, the values from Fig. 2 for the old

*M. O. Withey, "Test of Reinforced Concrete Columns," University of Wisconsin, Bul. 466, 1910.
†These formulas were later revised on the floor of the Convention. The costs given for the 1936 report in Fig. 3 are based on the original formulas and are lower than those resulting from the revised formulas.

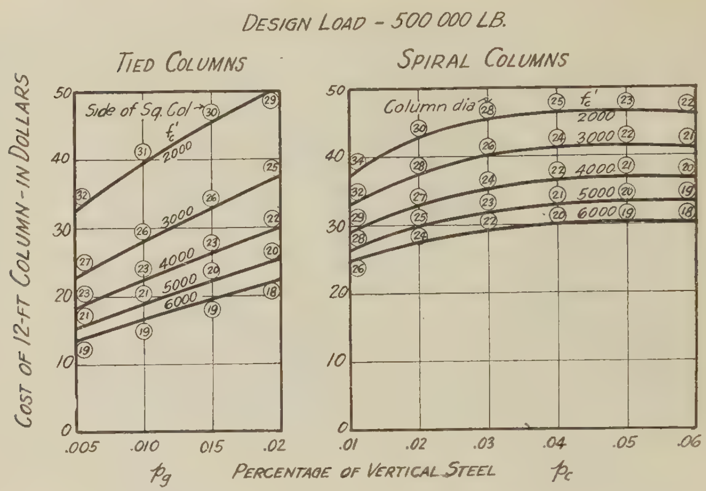

FIG. 2—COST OF TIED AND SPIRAL COLUMNS UNDER 1928 CODE: LOAD, 500,000 LB.

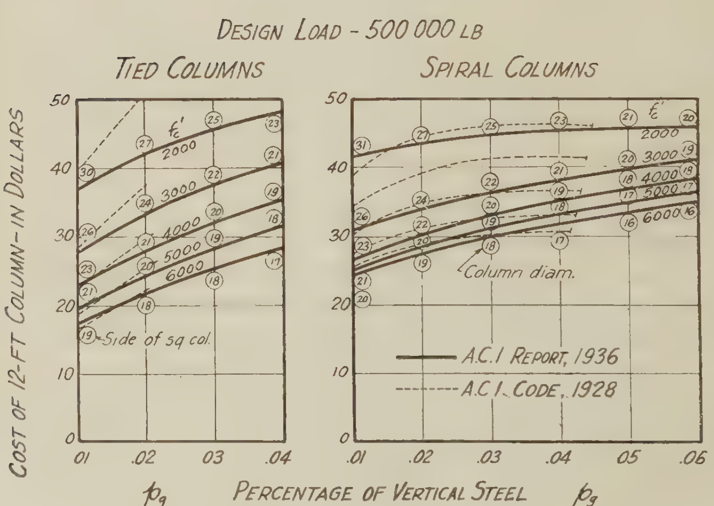

FIG 3—COST OF TIED AND SPIRAL COLUMNS UNDER 1936 REPORT AND 1928 CODE: LOAD, 500,000 LB.

A. C. I. code, are also shown in dotted lines on this figure, due account being taken of the different basis of calculating steel percentages in the two cases. It is seen that the 1936 formulas resulted, in general, in lower column costs, as well as in smaller columns, than those obtained with the 1928 code. For spiral columns, the 1936 costs for 2000-lb. concrete are held unusually high by the requirement of a minimum spiral percentage of 1⅛ per cent. With the stronger concretes the spiral cost still remains relatively high, but the reduction in concrete area and in vertical steel are enough to produce very marked cost reductions as the concrete strength is increased. The balance between tied and spiral column design is better than in the old code, but the tied column still shows a considerable margin in economy for a majority of the possible designs covered.

The possibility of saving in column costs may be examined by considering both tied and spiral columns carrying 500,000 lb. load under both codes, using as a representative percentage of vertical steel a value of 2 per cent of gross area. The relative costs are as follows:

Concrete Strength	1928 A. C. I. Code		1936 A. C. I. Report	
	Spiral	Tied	Spiral	Tied
2000	100.0	100.0	100.0	100.0
3000	87.8	73.4	78.0	78.7
4000	77.3	59.0	69.7	65.9
5000	69.6	49.7	65.4	57.0
6000	64.1	43.5	62.7	51.0

As a generalization of these data, giving principal weight to the values for tied columns, which represent by far the greatest number of columns used in buildings, the possible saving through the use of concretes stronger than 2000 p. s. i. might be summed up in the following average figures:

for 3000-lb. concrete, 20 per cent; for 4000-lb. concrete, 33 per cent;
for 5000-lb. concrete, 42 per cent; and for 6000-lb. concrete, 50 per cent.

These rather large savings in the cost of columns under axial loading will be reduced considerably if such columns are also subjected to bending stress, as is frequently the case with exterior columns.

FOOTINGS

In the design of footings, bond is normally an important consideration; however, under the A. C. I. code, limitations due to bond stress are eliminated through the device known as special anchorage. By hooking the ends of footing bars, the allowable bond stress on the bars is doubled. The design of footings under the code is therefore governed by flexure and diagonal tension. This being the case, the possible saving due to strong concretes falls in about the same class

as that for the rectangular beam of varying depth and constant width. The principal savings through the use of high strength concrete in footings will be through the reduced load transmitted to them. This may result in a smaller footing, or in the use of a less expensive type of footing. Some typical footing costs were quoted some years ago by F. E. Brown, writing on "Chicago Foundations" in the Handbook of the Illinois Section, American Society of Architects. He states that to carry a load of 700,000 lb., a spread footing costs about 89 cents per thousand pounds carried, a pile footing about $1.20 per thousand pounds carried, and a rock caisson to elevation −95 ft. about $2.31 per thousand. It is evident that a reduction in load which would permit the substitution of spread footings for pile footings, would effect a very considerable saving in foundation costs.

SUMMARY OF DIRECT COST REDUCTIONS

A summary of the savings to be effected in the different parts of the building structure may be made by a consideration of the relative importance of each element. A study of building plans indicates that of the total cost of the bare concrete structure, the floors and roof will cost about two-thirds; the columns, 20 to 25 per cent, and the footings most of the remainder. As previously noted the saving in the building frame is confined almost entirely to the columns and footings. The savings on flat slabs or on slab and girder construction will be of the order of ± 2 to 3 per cent, except for cases of beams in which a reduction of beam width is possible. Even with the latter it is not likely that a floor system could be reduced in cost more than 5 or 6 per cent; this would mean 3 to 4 per cent of the total cost. In addition the reduction in load to columns and footings may amount to 4 to 12 per cent. This will result in a corresponding saving in the footings, but will not amount to more than 1 per cent of the total cost. The combination of load reduction and the material saving on columns may reach 25 to 55 per cent, or 6 to 14 per cent of the total cost of the structure. Summing up, the total saving on the entire structure for the range in concrete strength from 2000 to 6000 p. s. i. may reach values of 8 to 18 per cent. For heavy flat slab structures in which spiral columns predominate, the possible saving will be less, probably not over 13 or 14 per cent. It is probably unwise to attempt to generalize further as to possible savings; the application of the foregoing data to specific structures, however, should furnish a fairly definite idea of what reduction in cost is possible.

ADVANTAGES AND DISADVANTAGES OF HIGH-STRENGTH CONCRETE

Aside from the question of direct costs, there are many other properties of high-strength concrete which produce more or less tangible

advantages and disadvantages. In general, greater durability may be expected with the stronger material, the resistance to wear or abrasion is known to increase with the strength, and greater workability and ease of placing will usually be obtained during construction. In shrinkage, the richer material shows at a disadvantage, since a 6000-lb. concrete will shrink definitely more (in amounts varying from 25 to 75 per cent) than a 2000-lb. concrete. This shrinkage may not be a matter causing much difficulty in design, but must be listed as a liability and not an asset. Large amounts of shrinkage and volume change may require the use of expansion joints in a structure where they would not be needed under ordinary conditions. The property known as time yield, creep or flow of concrete, unlike shrinkage, does not appear to vary appreciably with variations in concrete strength, provided the stress applied is in all cases the same proportion of the concrete strength; this is the case under A. C. I. code stresses. The Institute's recent column investigation showed no particular difference in time yield between nominal 2000 and 5000-lb. concretes.*

The deflection of reinforced concrete members, a quantity not easily computed, is of interest as the depth of beams and slabs are decreased through the use of stronger concretes. The flat slab provisions of the code evidently are intended to provide a constant stiffness when different values of f'_c are employed; actually they fall short, both because the modulus of elasticity is not proportional to strength for strong concrete and because the effect of the reinforcing steel is not considered. The result, then, is that for flat slabs, and even more so for slabs and beams of varying depth, the deflection increases as stronger concretes are used. In beams of constant depth and varying width, the deflection remains more nearly constant than in the other cases. This increase in deflection, since it involves no increase in tensile strains and hence no increase in tensile cracks, should not be of great importance, though for various reasons some definite limit on deflections and a minimum value of slab thickness should be set.

The lack of validity of the relation $E_c = 1000 f'_c$, has been mentioned; a similar lack of complete proportionality exists with regard to compressive and bond stresses, for which tests indicate a somewhat lower ratio to cylinder strength for strong concretes than for weak ones. The slightly lower factor of safety for compressive and bond stresses for strong concretes indicate that caution is needed in applying a rule for working stresses to extreme limits not contemplated when the rule was orginated. However, so far as compressive strength is concerned, the factor of safety for code stresses is ample, being approximately 3.5

*See Bul. 237, University of Illinois, Eng. Exp. Sta., 1934.

as compared to a value of 2.25 to 2.5 for tensile steel. The code values of $f_c = 0.4f'_c$, and $0.45f'_c$ at sections of negative moment, are still conservative.

A considerable advantage to be derived from the use of stronger concretes is found in the possibility of longer spans and of greater clear floor space. The use of greater spans and larger panels requires co-operation between architect and engineer; however, there is no question of their desirability from the user's standpoint. Similarly, the reduction of column sizes will remove an oft-repeated objection to the taller concrete buildings, and floor areas in which the column space is held to a minimum should certainly receive preference from a rental standpoint.

HIGHER STEEL STRESSES

It has been demonstrated that flexural members, which comprise a large portion of any building, show little advantage through the use of strong concrete because although the depth of beam or slab may be reduced, this is offset by a required increase in reinforcing steel. Obviously, if the increase in concrete quality could be matched in steel quality which would justify a higher steel stress, a great saving in cost would result. Referring to Table 1, for flat slab design, with a 20-ft. panel under 300 lb. per sq. ft., it is easy to see that while the use of 4000-lb. concrete produces a saving (as compared to the 2000-lb. design) of only 0.3 per cent with the usual steel stress of 20,000 p. s. i., an increase to 25,000 p. s. i. would result in no increase in steel over the 2000-lb. design, and a saving of 9.5 per cent on the total cost. Similarly an increase to steel stress of 30,000 p. s. i. in the same case would produce a saving of 15.5 per cent in total cost. These values assume that there is no increase in cost of the steel used with the higher stresses.

While the possible savings are substantial, there may be several objections to the higher steel stresses. Without question there will be an increase in tensile strains in proportion to the higher stresses, with an accompanying increase in tensile cracks. This enlargement of tensile strain will also produce definitely greater deflections. High steel stresses produce a problem in higher bond stresses, without the corresponding increase in capacity which depends solely on concrete quality. The increase in size and number of cracks may affect both bond and shearing resistance of a beam, besides possibly permitting rusting of the steel in cases of severe exposure. Because of these several rather definite objections, the wisdom of increasing the stress in tension steel is far more doubtful than that of utilizing higher concrete quality. Raises in steel stresses should come slowly, and be guided by both experience and tests.

CAN POSSIBLE SAVINGS BE ATTAINED

Granting that a saving in 10 or 15 per cent of structural cost is possible, can it be attained under present practice, and should the owner of a building be advised to use high strength concrete? This is a question for structural engineers to consider. Under a system wherein neither architect nor structural engineer will take responsibility for thorough and adequate inspection (often because of insufficient fees) the use of unusually strong concrete should not be attempted. If the structural engineer is satisfied to design buildings that he never sees built, he should stick to his time-honored 2000-lb. concrete. However, it would seem that even a 10 per cent margin would pay a good inspection fee several times over and there should be some way for the owner to secure this inspection, and to realize a substantial saving at the same time.

GAIN OR LOSS TO THE CONCRETE INDUSTRY

Aside from the saving that has been indicated above as due to the owner of the building, is the building of high strength concrete otherwise desirable to the construction industry? In practically all cases, the savings indicated with increased concrete stresses have been effected through a large reduction in concrete quantities, much greater than will be compensated by increase in unit price. In the case of flexural members this has been accompanied by a marked increase in steel quantities. In columns, much of the saving in cost has been at the expense of the concrete quantities, though with the great latitude in amount of steel permitted, some designs show far less steel tonnage than others. Because of the relative importance of beams and slabs, it may be concluded that high strength concrete will result in a distinct reduction in total value of concrete used, combined with an increase in steel tonnage. However, if this results in a superior structure at a reduced cost, the result in a competitive market should be an expansion in volume of building in this type of material, so that the reduction in cost of a single job may be greatly overbalanced by the larger number of jobs to be handled.

ACKNOWLEDGMENT

This paper was originally prepared (in somewhat different form) at the request of the Ready Mixed Concrete Association and presented at their 1935 Convention. For its release to the Institute, as well as for many suggestions and for information on concrete costs, credit is due H. F. Thomson, President, and Stanton Walker, Secretary of the Association. The author is also indebted to W. S. Thomson for data on steel and fabrication costs. This information was obtained while

the NRA code was still in effect, and while market prices of steel have not changed appreciably since, it appears that actual prices have lowered. The effect, so far as the conclusions of this paper are concerned, is to permit slightly greater savings than have been conceded herein; such savings increase as the percentage of reinforcement increases.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1936. Such discussion should reach the Secretary by July 1, 1936.*

Recent Developments in the Manufacture and Use of Cast Stone*

BY C. G. WALKER†

During the last eight years it has been my privilege to be very closely associated with the cast stone industry. The paper which I am presenting here is based on personal experiences and observations in working with both the producers and the users of cast stone.

It is the avowed object of the American Concrete Institute "to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge." On that basis it is appropriate that through the Institute some of the misinformation and prejudices surrounding cast stone should be torn away, the real problems of the material studied, and the progress of the art recorded.

The extent to which even many concrete and cement men fail to comprehend the real character and proper function of cast stone is surprising. Most of *you* probably are accustomed to thinking of cast stone as an imitation of one or another quarried stone, and you, more than likely, have a notion that in all cases it is or should be less expensive than the material it displaces. Unfortunately, this same notion has also been held by altogether too many of those who make and those who use cast stone. As a direct consequence of this attitude, the material has been forced to play the role of an imitative substitute with the inevitable result that its performance has frequently been unsatisfactory.

The increasing attention *now* being received by concrete as a full-fledged architectural material gives new importance to this much maligned and misunderstood product, cast stone. *Cast stone is concrete.* It is disregard of that fact and the consequent violation of fundamental principles of concrete by those who make and use cast stone that are responsible for most of the difficulties with it. The development of concrete architecturally can be greatly facilitated by the appropriate utilization of cast stone in its true character as precast concrete. The

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purpose of this paper and of the exhibition in the adjoining room is to supply information on developments in the manufacture and use of cast stone and its inherent possibilities as *precast architectural concrete.*

IMPROVEMENT IN PHYSICAL QUALITY

One of the most important recent developments in cast stone has been the remarkable improvement in its physical quality. Prior to the adoption of a tentative specification by the American Concrete Institute in 1929, neither users nor producers of cast stone were accustomed to pay much attention to strength and absorption. As pointed out in the first report of Committee P-3, all the emphasis was being laid on architectural details at the expense of those properties upon which the durability of the material depended. In the few instances in which physical quality was considered a compressive strength of 1500 p.s.i. was specified. When Committee P-3 set 5,000 p.s.i. as the minimum requirement for cast stone there was a great deal of head shaking among cast stone manufacturers and within the concrete fraternity. Today that requirement is being met readily by most manufacturers and a number of them are advocating requirements considerably higher.

TABLE 1—COMPRESSIVE STRENGTH OF CAST STONE

Samples selected at random. Tests made on 2-inch cubes
Specimens dried at 105°C. before test

Lab. No.	Compressive Strength lb. sq. in.	Source of Sample Tested
223	9490	Campbellsport Bank Bldg., Campbellsport, Wis.
301	9130	Colored Panel from manufacturers yard (Exhibit 16)
364	9540	duPont Carillon Tower, Nemours, Del.
365	9760	duPont Carillon Tower, Nemours, Del.
366	7380	High School Bldg., Henderson, N. C.
370	10860	Oceanside High School, Oceanside, Long Island
372	10950	Store Building, 2158 N. Clark St., Chicago
374	9100	Manufacturers' yard, New Orleans, La.
376	8900	Village Hall, Floral Park, Long Island
377	9000	Art Museum Building, Wichita, Kansas
378	9700	Genesee Brewery Bldg., Rochester, N. Y.
379*	9240	Barney Balaban Swimming Pool, Plano, Ill.

*3-inch cubes.

In a recent survey of quality, samples of cast stone were taken from actual jobs and tested. The results are given in Table 1.

These samples came from different plants representing all the commonly used processes of manufacturing, and they were in no way specially treated. From these test results it is apparent that the present requirements of the tentative specification are far below the quality of much of the cast stone being produced commercially. To that extent the present specification does not assure to the purchaser the benefits of the recent improvements in cast stone quality and in fact may stand in the way of his getting better cast stone. Under the

circumstances it appears that the requirements of the specification might well be raised to a point more in line with the upper level of quality.

Some questions have been raised as to why such emphasis should be laid on compressive strength in cast stone in view of the fact that even a low compressive strength would carry the loads to which the material is normally subjected. One very good reason is that compressive strength is a reliable measure of the skill and care used in the manufacture of cast stone. The nature of cast stone work is such that it requires a mix which is rich in comparison with what is used in most other forms of concrete. Mixes for cast stone range generally between 1:3 and 1:4 by weight and inexperienced manufacturers are very apt to use even richer mixes. On the basis of such mixes, high strength is the natural result of the application of reasonable care and thoroughness in manufacturing processes. It is, in effect, a by-product of skill and experience. The requirement of high strengths is one means of making sure that manufacturers will be vigilant and thorough in their plant processes. A lower requirement gives encouragement to laxness and to back-yard operations.

There is another good reason for high standards of quality in cast stone. Cast stone is not only concrete but by its very nature it is, or should be, concrete at its very best both in form and quality. It is made under conditions which permit close control of materials and handling if the manufacturer is so minded. There is little point to having concrete made in a plant unless the benefits of the possibilities of plant production as contrasted with field production are to be obtained. Setting high standards of quality for cast stone requires it really to be what it may properly be expected to be.

EXPOSED AGGREGATE FINISHES

In another development in the cast stone field which has vast significance, manufacturers have gone back more than twenty years for their technique. I refer to the increasing production of the type of cast stone having a finish in which the aggregate is exposed by wire brushing or washing with an acid solution. While this can scarcely be classified as a recent development, its increasing application is noteworthy as an indication of the growing tendency to treat cast stone as concrete and to give it finishes which are natural to its character. This type of finish belongs exclusively to concrete and cast stone and more than any other it fits into the scheme of concrete as a plastic architectural material.

The technique involved in this type of exposed aggregate finish has long been known. Mr. Whipple explained it fully in the book which

he published in 1915* but most cast stone manufacturers failed to realize its possibilities until concrete began to win recognition as an architectural medium in its own character.

GAP GRADING

As a basis for finishes of this type, cast stone manufacturers are learning to apply information on aggregate grading which is also old. In the November 27 issue of *Engineering Record*, an article by Robert H. McNeilly, assistant professor of Civil Engineering at Vanderbilt University, gives the results of his experiments with what he termed a "jump" grading in aggregate for mortar and concrete. Professor McNeilly found that by eliminating sizes intermediate between the coarse and the fine the largest possible amount of aggregate could be crowded into the concrete and its strength and density greatly improved. For a 1:3 mix, using an aggregate of ¼-in. as maximum size he specifically recommended a grading of 53 per cent passing the No. 4 and retained on the No. 10 sieve and 47 per cent including the cement, passing the No. 40 sieve. On the basis of aggregate alone this grading becomes 70 per cent of coarse material and 30 per cent of fine material. Many of the samples in the present exhibition are made according to this principle.

While Professor McNeilly purposely limited his experiments to mortars, his results led him to conclude that the same principles could be applied to regular concrete mixtures with equally beneficial results. In so far as the appearance of cast stone and of all other forms of architectural concrete is dependent upon the aggregate, this theory of grading as established by the experiments of Professor McNeilly is of special importance.

AUTOMATIC MACHINE MOLDING

In the field of manufacturing practices, the application of automatic machine molding to the production of cast stone is especially interesting both in itself and as a reflection of the study constantly being given by progressive manufacturers to the improvement of plant practices. Machine molding has been in actual use in the plant of the Dextone Company, New Haven, for more than four years. The best possible proof that it really works is provided in the fact that a second installation was made after the first one was destroyed in a fire.

In the process of machine molding (Fig. 1) a pattern is fastened face up to a pallet which in turn is fastened on the platen or molding platform of the machine. Sideforms are set around the pattern to the necessary height, forming what is known as a flask. Molding

*"*Concrete Stone Manufacture*" (out of print).

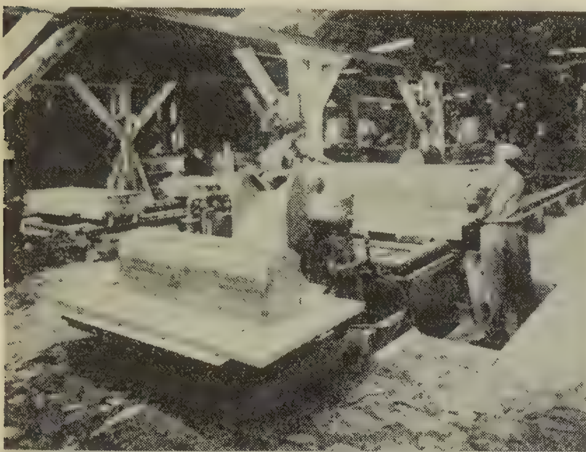

FIG. 1—PATTERN FACE UP ON PALLET ON PLATEN OF MOLDING MACHINE

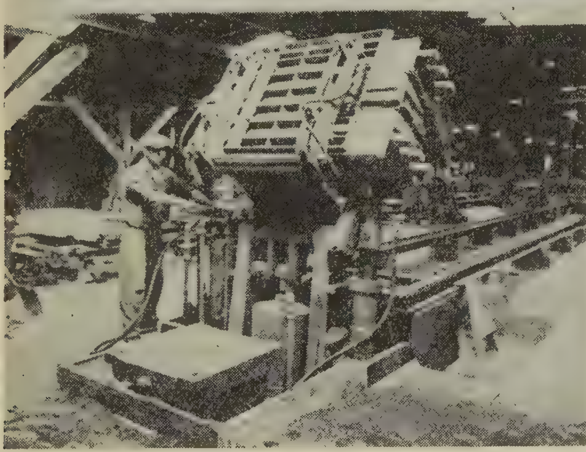

FIG. 2—PLATFORM OF MACHINE IS ROLLED OVER

sand is then fed into the flask from an overhead bin and packed around the pattern by jarring of the molding platform. After the flask is jarred the sand is struck off and a heavy bottom board is clamped on. The platform of the machine is then automatically rolled completely over, (Fig. 2) depositing the flask upside down on the opposite side of the machine on a hydraulic elevator. The clamps are released and the molding platform is rolled back, (Fig. 3) automatically withdrawing the pattern from the mold and returning it to the original position ready for another mold to be made. It requires approximately 10 seconds to jolt the sand into the flask, 15 seconds to roll the mold over, and 10 seconds to withdraw the pattern and roll it back. The mold shown on the machine in the pictures makes a piece of cast stone weighing about 800 pounds. At least 20 minutes would have

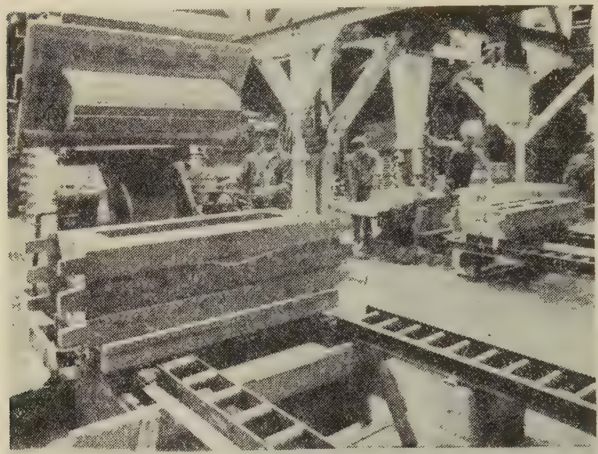

Fig. 3—Pattern withdrawn from mold

Fig. 4—Molds on conveyor to be filled

been required to make this mold by the old method of hand tamping the sand around the pattern. (See foreground Fig. 6). The machine will handle a load of 2600 pounds which with allowance for the sand and the flask means that a piece of cast stone containing about 15 cubic feet can be handled. The table will accommodate a single piece four feet wide by eight feet long, or a number of small molds can be made in the same flask.

The completed molds are carried from the machine to the filling station by heavy roller conveyors on which the cast stone remains until it is finished and stacked ready for shipment. (Fig. 4 shows a group of molds on their way to the filling station.) Within 24 hours the cast stone is removed from the sand and distributed to the finish-

FIG. 5—MOLDS FROM WHICH CAST STONE IS TO BE REMOVED

FIG. 6—THE OLD METHOD

ing room by means of turn tables intersecting the separate lines of the roller conveyor system. (Fig. 5 shows molds from which the cast stone is about to be removed.) The flasks are returned to storage behind the molding machine and the sand automatically drops into an elevator pit from which it is reconditioned and elevated to bins directly over the molding machines.

The advantages of machine molding lie in reduction of labor costs, increase in speed and capacity of production, and improvement in physical quality and appearance. A large part of *all* this is due to the accuracy and precision of the machine-made mold itself which eliminates the trouble and expense of correcting the imperfections previously experienced under the old system of making molds in

sand beds with the pattern face down and the sand tamped around it by hand.

It is developments like this that are taking the production of cast stone out of the status of a back-yard business. A great deal of credit is due to the enterprising manufacturers who persist in their efforts to improve the product in spite of the slowness with which the results of their efforts are recognized by those who use cast stone.

VIBRATED CAST STONE

While vibration is being applied to the production of cast stone, its adoption has not been as widespread as might be expected. This has been due chiefly to the failure or unwillingness of users to recognize that vibrated cast stone has definitely superior qualities which justify its cost. At the same time, even though the benefits of vibration are unquestioned, the new manufacturing problems which it introduces have been a serious obstacle to its wider application in cast stone plants.

Vibration does produce cast stone of markedly higher physical quality and on one hand it saves money for the manufacturer by permitting the use of leaner mixes and by reducing the cost of most finishes in which the aggregate must be fully exposed. On the other hand, it adds appreciably to the cost by requiring molds which are exceptionally strong and watertight and by either slowing up production or requiring a large number of duplicate molds to be made up. Under vibration, a very tiny hole or crack in the mold will permit leakage of water with resultant ragged edges or the pumping in of air with resultant air holes in the surface of the cast stone.

Probably the biggest benefit of vibration is that it makes polished cast stone possible. A durable polish can be applied to concrete only when the surface has a minimum of cement paste exposed. Through vibration it is possible to cover 80 per cent or more of the surface with aggregate particles. With less than this amount of coverage the luster of a polished surface will be noticeably dimmed as the exposed cement paste gradually loses its polish. Incidentally, aggregate of the granite type must be used in polished cast stone since other types of aggregate, and especially marbles, will not retain a polish under exterior exposure.

As in all other forms of concrete, the benefits of vibration are obtained only when the mix is specifically designed for vibration. In the mistaken belief that vibration is good in and by itself, some manufacturers vibrate the mixes which they customarily use in other methods of placing. As a matter of fact such an application of vibration is apt to give disappointing results. The first changes in adjusting a mix for vibration should be to take out a large percentage of the

finer aggregate, reduce the proportion of cement to aggregate and cut down the amount of mixing water. In this connection, manufacturers are learning that the recommendations of Professor McNeilly on "jump" grading of aggregate are particularly applicable to mixes which are to be vibrated. On the basis of ⅜-in. aggregate, which is about as coarse as is ever used in cast stone, excellent results are being obtained with a mix of 1:4½ using 4¾ gallons of mixing water. While a mix of these proportions with less water can be molded by vibration, considerable difficulty is almost sure to be encountered with air holes which are particularly objectionable in surfaces to be honed or polished.

Choice of vibrating equipment varies among manufacturers. One plant which has had excellent success with vibrated cast stone uses air vibrators of a type which is applied directly to the side of the mold and a small surface vibrator which is placed on top of the concrete within the mold. Another plant which produces vibrated cast stone exclusively uses vibrating tables. Internal vibration is not applicable to cast stone work due to the shallow molds.

In spite of the benefits to be obtained from it, vibration does not appear likely to displace other methods of molding cast stone until there is more discrimination in favor of high quality and until the physics of its application to concrete mixes is better understood.

CAST STONE FORMS FOR CONCRETE

The most outstanding recent development in the application of cast stone is its use as forming for structural concrete. While this represents a radical departure from customary practices in concrete construction, it is an entirely logical development in the growing use of concrete in architecture. Such a system of construction is especially applicable when it is desired to decorate monolithic concrete through the use of color pigments, white cement or special aggregate. The inclusion of any of these materials throughout the full thickness of a concrete wall would be economically impracticable in most cases but they could readily be used in thin slabs of cast stone and the cast stone in turn used as the combined form and decorative treatment for ordinary concrete. This method of construction also provides a practical way of building insulation into a concrete wall through the use of a special backing on the cast stone itself.

There are two general methods of using cast stone as forming. One eliminates all wooden forming on the outside face of the wall and depends upon the cast stone slabs themselves to carry the pressure of the freshly placed concrete. Such a system was used in the con-

struction of the 210 foot duPont Memorial Tower at Nemours, Del., during 1935.

The other general method of applying cast stone as forming uses relatively thin slabs held in line and supported by studs on the outside face of the wall which are tied through the joints to the inner form. This system was used in the construction of the City Hall in Santa Ana, Calif., during 1935. A similar method was used in constructing the $8,000,000 Dorchester Hotel in London in 1930 in which cast stone slabs were used as the outside forming and 2-in. cork boards served the double purpose of inside forming and insulation to which plaster was applied direct.

HOMOGENEOUS VS FACED STONE

In spite of what might be termed the "coming of age" of the cast stone industry there still remain some rather pointless controversies over production methods and practices which serve only to confuse those who use the material. One popular bone of contention is the relative merits or demerits of what is referred to as faced cast stone as compared with cast stone which is homogeneous throughout. As a matter of fact, cast stone is neither good nor bad simply because it is homogeneous or because it is composed of different materials in the facing and the backing. Tests have shown a range from extremely high to extremely low quality in both types. Reports on their records in service show them just about even on actual performance. While the Federal Specification on cast stone unfortunately implies a difference in quality by classifying homogeneous cast stone as Type I and faced cast stone as Type II, the physical requirements are the same for both. A tabulation of test results on specimens taken from actual jobs (Table 2) certainly gives no support for differentiation between faced cast stone and homogeneous cast stone on the basis of physical quality.

TABLE 2—COMPRESSIVE STRENGTH OF CAST STONE

Samples selected at random. Tests made on 2-inch cubes
Specimens dried at 105°C. before test

Lab. No.	Compressive Strength lb. sq. in.	Type
223	9490	Faced
301	9130	Faced
364	9540	Homogeneous
365	9760	Homogeneous
366	7380	Faced
370	10860	Homogeneous
372	10950	Faced
374	9100	Homogeneous
376	8900	Homogeneous
377	9000	Faced
378	9700	Faced
379	9240	Homogeneous

Expediency and economy are the real considerations which determine whether a manufacturer makes his cast stone homogeneous or faces it. Some methods of manufacture simply do not lend themselves to the practice of facing. Some manufacturers using such methods have attempted to make a virtue out of a necessity and have been successful in establishing the idea that homogeneous cast stone is by its very nature much to be preferred over cast stone that is not homogeneous. It should be clearly understood that this is a matter of personal opinion.

The practice of using different materials in the face and the body of cast stone is a perfectly legitimate device for reducing cost. The logic and value of the practice is evidenced by the simplest of arithmetic. Approximately 32 pounds of aggregate is required to produce a square foot of cast stone 4 inches thick. The substitution of a $4-per-ton aggregate for a $30-per-ton aggregate in 3 inches of backing will effect a saving of approximately 30 cents per square foot for aggregate alone. There would be additional savings in the event special cements or color pigments were being used. Savings on materials would of course be offset to some extent by the added expense of handling two different mixtures but the final result would be much in favor of the practice of using facing.

The most frequent criticism of faced cast stone is that the facing is likely to be split away from the backing. Ample proof is available from actual jobs that there is no inherent weakness in the bond between facing and backing. Broken test specimens demonstrate the total absence of a line of weakness between the two materials.* Whenever failure along this line does occur it may safely be taken as evidence of poor workmanship rather than as evidence of an inherent fault in this method of making cast stone. Unfortunately, it must be admitted that some manufacturers have been guilty of either just such carelessness in their work or plain ignorance of the fundamentals of their business.

MANUFACTURING PROCESSES

What has been said of homogeneous and faced cast stone applies equally well to claims and counter claims with respect to processes of manufacture. Cast stone is neither superior or inferior merely because it is made by a particular process of molding. There are four general methods in use: sand molding which uses a wet mix in absorbent sand molds; puddling which uses a less wet mix in rigid molds; tamping which uses an earth-moist consistency in rigid molds; and vibrating. Each of these methods has its advantages and its weaknesses. The

*Such test pieces were a part of the exhibit of cast stone.

tamping method in particular is frequently condemned as productive of nothing but inferior cast stone. It is a paradox that some of the best cast stone has been made by this method and at the same time some of the very poorest. This method is especially susceptible to abuse at the hands of manufacturers whose chief interest is in the lowest possible price. At the same time there are distinct advantages in the principle underlying the tamping process and it is likely that one of the next developments in manufacturing practices will be machine application of this method of molding.

TIME A FACTOR IN MAKING GOOD CAST STONE

Perhaps the greatest problem with which cast stone manufacturers have to deal grows out of the failure of most users to comprehend the importance of time as one element in cast stone making. It is safe to say that there is not one job out of twenty on which the manufacturer is allowed sufficient time to make and cure his cast stone properly to say nothing of seasoning it. Therein lies the cause of one of the most common complaints against cast stone—the opening of mortar joints with resultant discoloration or leakage.

The material has been most unjustly condemned because like practically every other solid body it contracts with loss of moisture. This phenomenon of shrinkage upon drying is the reason why lumber is kiln-dried before it is used, why careful architects specify that limestone shall be thoroughly seasoned and free from quarry sap, and why the terra cotta manufacturer uses in his shop a rule which has 12¾ inches to the foot. Yet cast stone is set before it has had any chance at all to dry out and then roundly dammed for the occurrence of a most natural phenomenon.

There is urgent need for recognition by both the producers and users of cast stone that a period of seasoning is just as important as a period of curing. One of the next developments in manufacturing practices might well be a method for accelerating and improving the curing and seasoning process but the need for it can never be eliminated as long as water is used in the mixing of the concrete.

There are other problems in the manufacture and use of cast stone which time does not permit discussing here. It is neither a perfect nor a fool-proof material. To those scientists and researchers among your membership who are looking for new worlds to conquer, cast stone, the Cinderella of concrete, offers a field of study which is both varied and interesting.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by July 1, 1936.*

Tests of the Resistance of Concrete Masonry Walls to the Penetration of rain*

BY R. E. COPELAND† AND C. C. CARLSON†

LEAKY masonry walls persist today as one of the most perplexing problems in the construction of masonry structures. It is intensified by the non-homogeneous character of unit masonry. The materials themselves are more or less porous and will admit moisture. When combined in a wall, they provide separation planes at the bed and head joints where moisture may seep through. Fortunately, the amount of moisture admitted by the wall during rains of light or moderate intensity usually is insufficient to cause penetration to the inside face. However, in most sections of the country, hard driving rains of long duration occasionally occur and cause many cases of leaky masonry.

It is a well known and experienced fact that no type of masonry not especially surface sealed is immune to this trouble nor is leakage any respector of the size or cost of the structure. Some of the finest buildings having unit masonry bearing or enclosing walls have leaked so badly a few months after completion that extensive waterproofing and repairs were required.

TESTING RAIN RESISTANCE

In view of this existing situation, the research described herein on rain resistance of masonry walls has been rather broadly planned with regard to types of materials and construction. That part completed to date and which is briefly reported in this paper, deals entirely with unpainted and painted walls of concrete masonry and has involved 45 different test specimens and a total of 91 tests. The purposes of this part of the research were to determine.

1. The rain endurance period of the walls when exposed to selected and controlled intensities of wind and rain in combination;

2. The effect of variations in the physical properties of the masonry materials and in the construction and exposed surface treatment of the wall; and

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936.
†Development Department, Portland Cement Association.

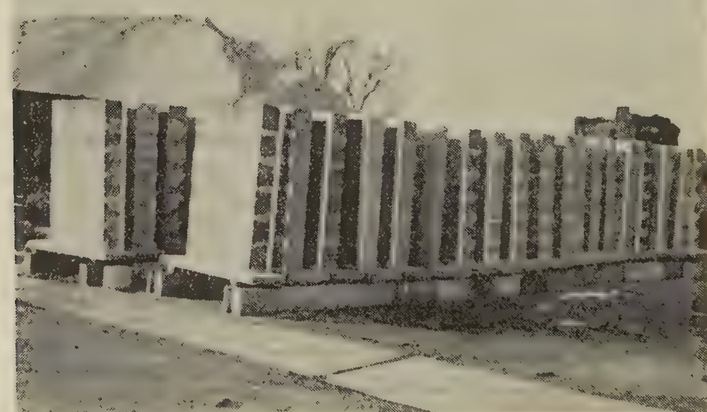

Fig. 1—General view of test shed, constant head storage tanks and two walls in position ready for test

Fig. 2—Group of test walls

3. To develop information which may be applied in construction practice to insure that walls of this type will not leak under extreme rain conditions.

Description of Test Walls

The test walls were 32 in. wide by 48 in. high. The joints were about ⅜ in. thick. The unit designs included 8 by 8 by 16 in. 3-oval core plain face and water-eroded face block with 32.3 per cent core area, 4 by 8 by 16 in. 3-core plain face partition tile with 24.3 per cent core area, and 8 by 8 by 16 in. 3-oval core oscillated face block with 40

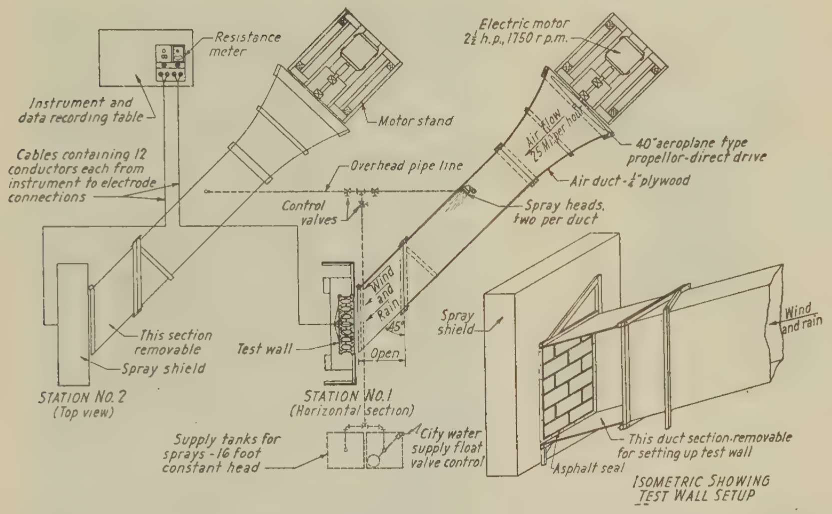

FIG. 3—ARRANGEMENT OF APPARATUS FOR RAIN RESISTANCE TESTS

per cent core area. Both lightweight and heavy types of aggregates were used. The block mixes varied from 1:6 to 1:11, based on dry rodded volume aggregate measure. There were 4 aggregate grading fineness moduli between the limits of 3.50 and 4.50. Mortars included 1:3 lime, 1:1:6 cement-lime, and 1:0.25:3 cement-lime. The mortar sand graded 2.42 F.M. Most of the walls were built with face shell bedding, but some had full mortar bedding.

The block and tile units were made in commercial plants in power tamper, stripper machines. All except the oscillated face units were steam cured 24 hours, then stored in air under shelter. The oscillated face units were made in a different plant which did not have steam curing facilities. These units were damp cured 3 days, then stored in air under shelter. The machine for producing the oscillated face units was of a standard stripper type having oscillating plate attachments in the mold box which produced a trowelling effect on the face shell simultaneously with the tamping.

Four walls were built of sand and gravel water-eroded face units. The water-eroding treatment was accomplished with a fine garden hose spray applied to one face shell just before the concrete took its initial set and consisted in effect of washing out the cement and fine aggregate on the surface, leaving the coarse aggregate exposed.

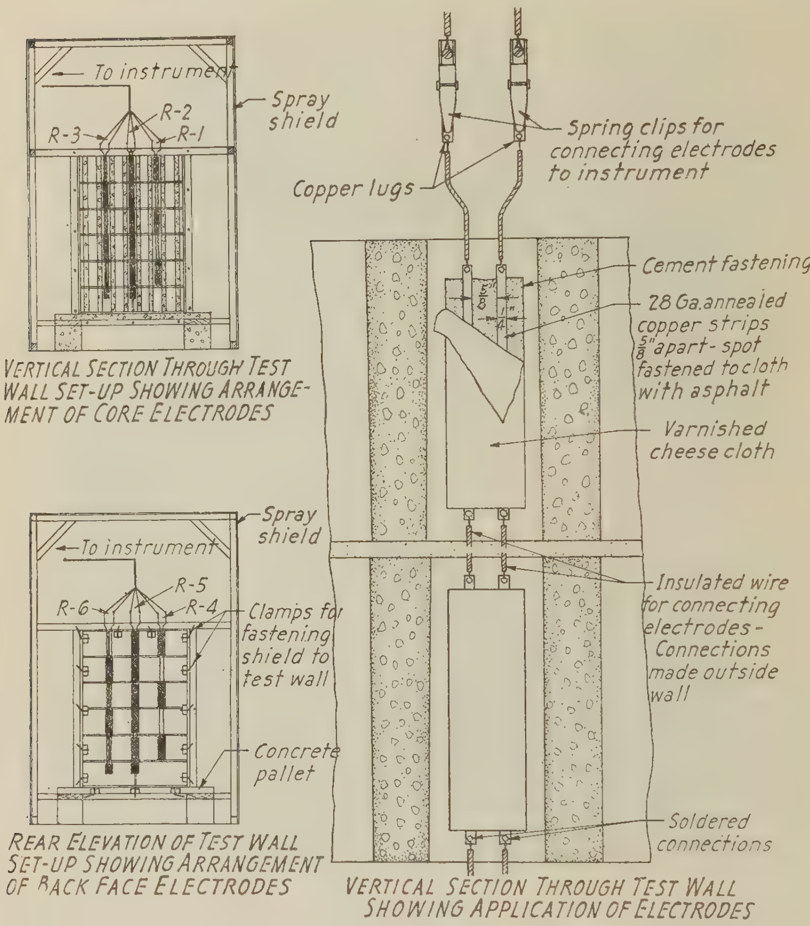

FIG. 4—ARRANGEMENT OF ELECTRODES

Wind and Rain Apparatus

Fig. 3 to 5 inclusive show the principal details of the apparatus developed to produce the simulated rain condition to which the test walls were exposed. The wind velocity was accurately measured as 25.3 miles per hour by a precision anemometer. The rain intensity used for the unpainted walls was 2½ to 3 in. per sq. ft. per hour. In testing the painted walls, this same intensity was used during the first 12 hours of the test, after which the intensity was increased to an accelerated rate of 12 to 14 in. per sq. ft. per hour for a second 12-hour

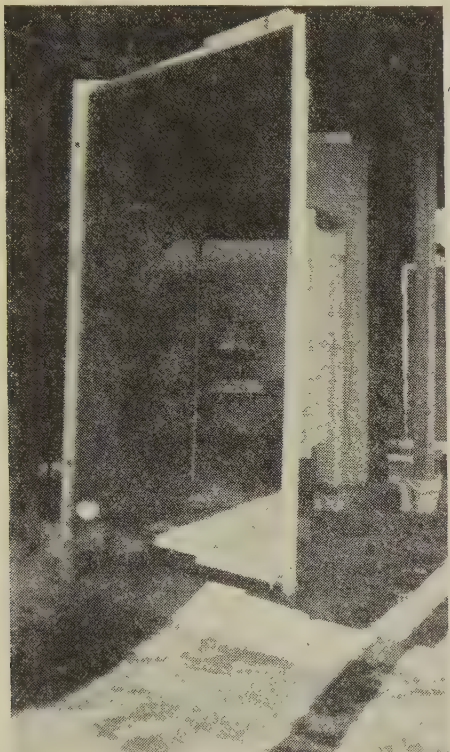

Fig. 5—Wind and rain duct showing water sprays and airplane type propeller

period of the 24-hour test in an effort to obtain failure. These intensities represent the actual impringement against the test wall as determined by frequent calibrations. The spray discharge was considerably greater but some water was lost in the duct and around the side of the wall.

The weather bureau records of excessive precipitations indicate that intensities of 2½ inches or greater seldom exceed 60 minutes in duration. The test walls, therefore, were subjected to a rainstorm of considerable severity.

Method of Measuring Wall Performance

Rain penetration through the face shells and back face of the test wall was detected by frequent visual examinations supplemented by resistance readings resulting from passing constant voltage current through copper strip electrodes of which there were three series arranged in the core spaces back of the exposed face shells and three series on the back face of the wall. (See Fig. 4). Initial resistances for

the dry wall ranged from 200,000 ohms to 5 megohms, but these dropped abruptly when any one electrode in a series became affected by moisture. A reduction to 5,000 ohms or less definitely indicated that the concrete had become visibly damp somewhere under an electrode in a series. Readings as low as 500 to 100 ohms were obtained as the concrete became wetter. Since each set of electrodes was connected in a series, a damp spot under any one electrode registered failure for the entire series.

Test Procedure

There were duplicate sets of rain and wind apparatus and usually two walls were tested simultaneously. Before the rain and wind were turned on, initial electrode readings were taken of the dry concrete. After the test was started, readings were taken and visual examinations made as frequently as conditions required, usually at five-minute intervals for the first hour and 10 to 30 minute intervals thereafter, depending on the performance of the wall.

Each test was continued until appreciable damp areas appeared on the back face and the back face electrode resistances registered failure, except that in the case of the painted walls, the test generally was discontinued after 24 hours (12 hours, 2½- to 3½-in. rain; 12 hours, 12- to 14-in. rain), even though the wall had not failed. A few specimens were tested for longer periods in an effort to obtain failure.

The recorded observations included the electrode resistance readings, the time of exposure required to produce initial visible penetration of the face shells and back face, and also the sources of leakage as nearly as could be determined. The endurance period of the wall was taken as one-half the sum of the time intervals required for the first two back face electrode failures. The average of the three electrode series was not used because frequently one of these would hold out an unreasonable time because of the erratic nature of back face leakage as to location.

PERFORMANCE OF WALLS

The test results indicated that the various walls divided into two performance groups: the unpainted walls showed relatively low resistance to rain penetration in comparison with the practically impervious qualities of the painted walls.

Leakage was observed to take place through (a) the mortar, (b) the joint between mortar and masonry unit, and (c) the concrete of the masonry unit.

Face shell penetration was largely due to the dynamic action of the wind and rain while penetration of the back face appeared to be a

process of forced permeation and capillary absorption influenced by the qualities and peculiarities of the materials and construction of the wall. Initial penetration occurred at the weakest points of the wall in time intervals which were frequently inconsistent with the average properties and resistance of the wall. In general, the performance of the unpainted walls appeared to be greatly influenced by any small, presumably unimportant, defects which were present in either the units or mortar joints. The performance of the painted walls was largely determined by the effectiveness of the paint coatings.

Unpainted Walls

Considering the 8-in. unpainted walls, the initial penetration of the face shells usually occurred in 5 minutes or less and proceeded with such rapidity as to cause pools of water to form in a relatively short time in the core bottoms on the concrete pallet. The initial visible penetration of the back face was noted in from 10 minutes to 1 hour. Endurance periods, based on the failure of the first two back face electrodes, ranged from 15 minutes to 2 hours, 47 minutes with plain face block; from 1 hour, 18 minutes to 5 hours 10 minutes with oscillated face block made of light-weight aggregates; and from 18 minutes to 1 hour, 15 minutes with water eroded face sand and gravel block.

The walls built of 4 inch partition units leaked even more rapidly than the 8-inch walls, initial visible penetration of the back face occurring in from 3 to 5 minutes.

The results did not show any consistent differences of practical importance which might be ascribed to the variations of concrete mix, aggregate grading, or to the use of lightweight aggregates as compared with heavy aggregates. The oscillation of the face shells of the units made with lightweight aggregates retarded and reduced back face leakage. The water erosion treatment of the face shells of sand and gravel units seemed to facilitate leakage somewhat.

In most cases, the mortar or joint between mortar and unit leaked first but leakage through the concrete face shells frequently followed closely, especially with the more porous concretes. The mortar joint proved to be such a consistent source of early leakage that it tended to obscure the effects of other factors. An attempt was made to check the performance of selected walls previously tested with unpainted surfaces by retesting after painting over the joints on the exposed face but leaving the block face unpainted. The volume of leakage was reduced but early joint leakage was still an important factor. It appeared that moisture penetrating the concrete surface gravitated downward to the joint where it found an easy path to follow into the core space.

The effect of varying the types of mortars and joint construction was investigated in some walls built of lightweight aggregate concrete units. There was little difference in the results between the different walls. Possibly variation of mortars and joint construction would have had more pronounced effects if a denser, more impervious type of masonry unit had been used and no general conclusion was drawn.

Painted Walls

All walls in this series had been previously tested with unpainted surfaces. The paint used was a mixture of water and a commercial brand of white waterproof portland cement. Paint to be applied with a brush (a brush with short stiff fiber bristles was used) was mixed in the proportions of 1.6 pounds cement to 1 pound water. Paint to be sprayed on was mixed in the proportions of 2 pounds cement to 1 pound water. All except four walls were given two brush applied coatings. Two walls were given one brush coat and tested and then later retested after having received a second coat. Two walls were spray painted using standard spray equipment. The paint and air line pressures were 15 and 35 pounds respectively. The nozzle was adjusted to throw a conical spray which was directed at different angles onto the masonry.

Good workmanship was used throughout in all painting so as to cover the surface effectively and fill the voids. Each coat was applied on a damp base and was damp cured two days. At time of test (14 days after application) no crazing was noted in the brush applied coatings; a slight amount had developed in the sprayed on coatings.

The painted walls were subjected to 12 hours 2½- to 3-in. rain intensity and 25 m.p.h. wind followed by another 12 hours 12- to 14-in. rain intensity and 25 m.p.h. wind.

None of the 8-in., 2-coat brush painted walls showed any back face leakage. In no case was leakage through the face shells sufficient to cause pools of water to form in the core bottoms. The minimum time for initial penetration of the face shell was 2 hours, 45 minutes, and usually 4 hours or longer was required. Two walls showed no face shell penetration at the end of the 24-hour test.

One of the two walls tested with a single brush coat of paint failed by leakage through the back face in 22 hours 35 minutes. The other wall did not develop back face leakage. Both walls, however, showed greater penetration than when retested after receiving the second coat of paint.

In a test of one of the walls built of water eroded face units with two coats brush applied paint, sufficient penetration occurred as to

be on the point of breaking through the back face at the end of the test, thus indicating that the water eroding treatment has a slight detrimental effect, even in the case of painted walls.

The 4-in. walls brush painted two coats performed satisfactorily. The initial face shell penetration, as determined by the first electrode failure, occurred in from 1 hour 45 minutes to 4 hours 45 minutes but in no case did it progress sufficiently to cause pools of water to form in the core bottoms. The first visible leakage through the back face occurred in 2 hours 30 minutes at a joint in the worst case. For the other walls in this group, no back face leakage was observed to take place in less than 4 hours 45 minutes.

The wall with two coats of sprayed on paint developed face shell penetration in 25 minutes and the first visible back face leakage in 45 minutes. These time intervals for the walls with three coats sprayed on were 55 minutes and 4 hours 15 minutes respectively. The sprayed on coatings appeared to leak through a comparatively few pin holes which seemed to be inevitably left in the paint in spite of the use of coatings so heavy they tended to run.

Stuccoed Walls

One wall with 3 coats of portland cement stucco (1:0.25:3 cement-lime mixture) applied on lightweight aggregate concrete block was tested. It showed no signs of face shell or back face penetration after 219 hours of continuous testing consisting of 166 hours with the 2½- to 3-in. rain intensity and 53 hours with the 12- to 14-in. rain intensity.

CONCLUSIONS

The results of the tests completed to date point to the following conclusions pertaining to the rain resistance performance of concrete masonry when subjected to hard driving rains of similar intensity and duration to that employed for test purposes.

1. Walls properly painted with two coats brush applied waterproof portland cement paint will show no penetration through the back face and any penetration of the exposed face shells will be inconsequential. It would seem that existing unpainted concrete masonry structures could be effectively waterproofed with paint as well as new construction.

2. Good workmanship and proper methods are important factors in obtaining the full efficiency of paint coatings. Appreciable moisture will be admitted by small pin holes or breaks in the paint.

3. One coat of paint will greatly increase the rain resistance of unpainted walls but will be less effective than two coats. Two coat work is to be preferred over one excessively thick or double coat as the latter will have a greater tendency to craze.

4. The sprayed-on coatings were less effective than brushed-on coatings thus indicating the need for further development in the technique of spray painting.

5. Considering unpainted walls, the initial penetration of the exposed face shells will occur in 5 minutes or less, usually appearing first at the joints, and will progress with sufficient rapidity to cause pools of water to form wherever the core spaces are blocked off. Initial back face leakage will appear in one hour or less and will become widespread, usually in 2 hours or less.

6. Since the joints were a consistent source of early leakage in all of the unpainted wall tests it appears that the quality of impermeability in the units alone will not insure a rain resistant wall.

7. The performance of the stuccoed wall was noteworthy. It showed no signs of water penetration after 219 hours of continuous exposure thus demonstrating the practically impervious qualities of properly applied portland cement stucco on concrete masonry.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by July 1, 1936.*

Analysis of Multiple Span Rigid Frame Bridges by the Slope Deflection Method*

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PURPOSE OF PAPER

1. To present a concise and workable analysis for one-, two- and three-span continuous concrete rigid frame bridges by an adaptation of the slope deflection method. The author first presented this method in general form for members with a constant moment of inertia in 1915 (see Engineering Studies No. 1, University of Minnesota).

2. To present several complete numerical solutions for the statically indeterminate moment distribution which results with one-, two- and three-span rigid frame bridges made up of members having a variable moment of inertia using the method of analysis of Par. 1.

3. To present calculations and discussions regarding certain assumptions made in the solution of these illustrative examples of Par. 2.

4. To propose an analysis based on the use of construction joints designed to eliminate the high stresses resulting from temperature changes and rib shortening in the multiple span cases.

STATEMENT OF FUNDAMENTAL SLOPE DEFLECTION EQUATIONS

The general equation for members of constant cross section first presented in 1915 was in the form

$$M_{AB} = \pm M^F_{AB} + \frac{2EI}{L}(3R_{AB} - 2\theta_A - \theta_B) \quad \text{(a)}$$

M_{AB} = Final bending moment at the end A of any member AB.

M^F_{AB} = "Fixed beam" moment at same point which results from transverse loading applied to member when the ends are fixed against rotation ($\theta_A = \theta_B = 0$) and when the line between ends of axis of member is not allowed to rotate ($R_{AB} = 0$).

$\frac{EI}{L}$ = Product of modulus of elasticity and moment of inertia of member divided by its length.

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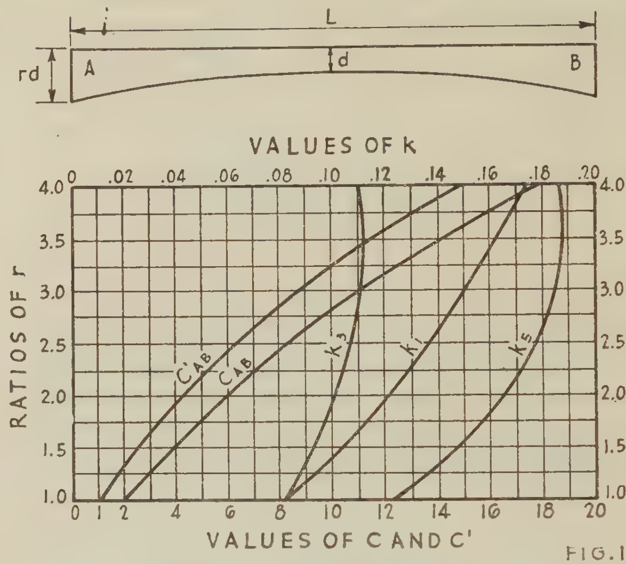


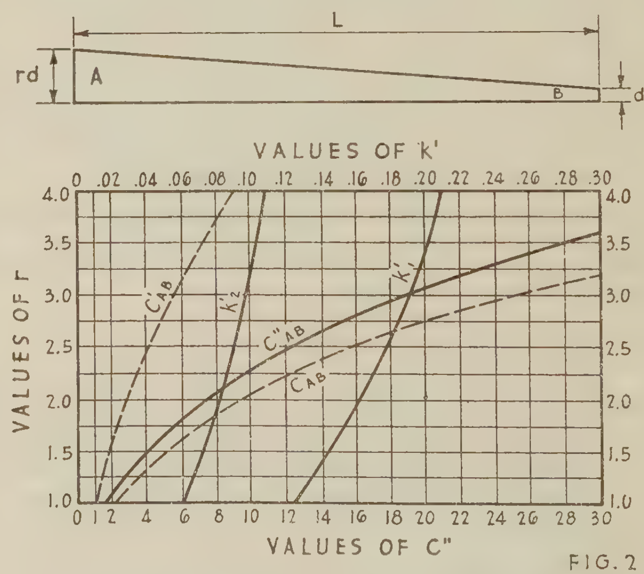


FIG. 1—LOAD AND BEAM COEFFICIENTS FOR PARABOLIC SLAB SPAN MEMBER OF RIGID FRAME BRIDGE

FIG. 2—LOAD AND BEAM COEFFICIENTS FOR PIER OR VERTICAL MEMBER OF RIGID FRAME BRIDGE

An examination of this equation shows that the final end moment is obtained from the fixed end moment by adding a correction in terms of Θ_A, Θ_B and the rotation of the member as a whole, R_{AB}. This fundamental idea of using fixed beam moments as a starting point for corrections to give the final bending moment in rigid frame analysis had its inception in the "slope deflection" method.

Moment distribution is a function of *relative* stiffness, so the form

$$M_{AB} = \pm M^F{}_{AB} + K_{AB}[3R_{AB} - 2\Theta_A - \Theta_B] \qquad \text{(b)}$$

is the more workable form where K_{AB} is a relative $\left[\frac{I}{L}\right]$ quantity, assuming $2E$ constant throughout. K_{AB} is generally the value of $\frac{bd^3}{L}$ for a reinforced concrete beam AB where the width b and the depth d may be used in inches and the span L in feet. It is convenient to call this K value for the member having the smallest $\frac{bd^3}{L}$ value unity, and dividing other $\frac{bd^3}{L}$ values by the smallest one to get the relative K values for other members of the frame.

When there is no rotation of lines between ends of a member in a rigid frame bridge, as when "side sway" is prevented in unsymmetrical loading, Equation (b) takes the following form

$$M_{AB} = \pm M^F{}_{AB} - K_{AB}[2\Theta_A + \Theta_B] \qquad \text{(c)}$$

The more general form of Equation (a) would be

$$M_{AB} = \pm M^{F'}{}_{AB} - K_{AB}[C_{AB}\Theta_A + C'_{AB}\Theta_B - (C_{AB} + C'_{AB})R] \qquad \text{(d)*}$$

In this form the "slope deflection" method is applicable to members of varying cross section, that is, with a variable moment of inertia. When "side sway" is prevented, the general equation (d) takes the form

$$M_{AB} = \pm M^{F'}{}_{AB} - K_{AB}[C_{AB}\Theta_A + C'_{AB}\Theta_B] \qquad \text{(1)}$$

PARABOLIC SLAB SPAN MEMBERS

The fixed end moment $M^{F'}{}_{AB}$ in Equations (d) and (1) for parabolic slab members may be computed by the formula

$$M^{F'}{}_{AB} = k_1 w_c L^2 + k_3 w_u L^2 + k_5 PL \qquad \text{(2)}$$

*"The Modified Slope Deflection Equations," by L. T. Evans, *Proceedings*, A. C. I., 1932, page 114, gives a prior statement of equation (d). The writer has developed it independently from the original equations for "Slope Deflection" as stated by him in 1915 and has used a slightly different form consistent with the original statement. Mr. Evans also published these equations previously.

w_c = weight per foot of slab at center of span
w_u = uniform live load per foot of slab span
P = concentrated live load at span center only
Constants C_{AB}, C'_{AB}, k_1, k_3 and k_5 of Equations
(1) and (2) may be taken from Fig. 1.

K_{AB} is found as before, except the *center* depth is used in the $\frac{bd^3}{L}$ equation. $M^F{}_{AB}$ is changed to $M^{F'}{}_{AB}$, and the standard coefficients of one-twelfth for uniform load and one-eighth for concentrated load no longer apply, but values of k_1, k_3 and k_5 for dead load, uniform live load and concentrated center live load, respectively, must be taken from the diagram of Fig. 1 for the design value of "r," which is the ratio of maximum to minimum depth for any member of varying depth. The coefficients k_1, k_3 and k_5 eliminate the need for determining these values with the help of influence lines. It should be noted that the concentrated load is placed at center span because the depth is minimum there. Somewhat greater "fixed beam" moments can be had by moving this load away from the center slightly. C_{AB} takes the place of the coefficient *2* for Θ_A in Equation (c), and C'_{AB} the place of the coefficient *1* of Θ_B. It will be seen that all values of k and C in the diagram of Fig. 1 have the well known values for constant moment of inertia case of Formula (c) when $r = 1$.

PIER MEMBERS

For the pier or post member with constant moment of inertia, Equation (c) takes the more convenient form of equation

$$M_{AB} = \pm M^F{}_{AB} - K_{AB} \times \frac{3}{2} \Theta_A \quad \text{(e)*}$$

This equation applies to members with a constant moment of inertia. This is a convenient form to use for a member where one end (bottom of pier member) is assumed to be hinged or to have zero bending moment under all conditions. The moment at the end A can then be expressed in terms of Θ_A only instead of both Θ_A and Θ_B. The number of joint equations is thus reduced by half for the rigid frame bridge. The more general expression for the case of the member of variable moment of inertia fixed at one end and hinged at the other would take the form

$$M_{AB} = \pm M^{F'}{}_{AB} - K_{AB}C''_{AB}\Theta_A \quad \text{(3)*}$$

where M_{AB} is the final end moment at A of member AB (shaped as shown in Fig. 2). The "fixed end" moment in Equation (3) may be computed by the formula

*$M^F{}_{AB}$ and $M^{F'}{}_{AB}$ in equations (e) and (3) are fixed end moments at A when A is fixed and B is hinged.

$$M^{F'}_{AB} = k'_1 w_1 L^2 + k'_2 w_2 L^2 + \frac{C''_{AB}\ (EI_B)\ \Delta}{72L^2} \qquad (4)$$

w_1 = Lateral earth unit pressure at A end of member

w_2 = Increase over w_1 of the lateral earth unit pressure at B end of member

w_1 and w_2 = Total earth pressure at bottom of member

EI_B = Modulus of elasticity of concrete $\times$ moment of inertia $\frac{bd^3}{12}\left[\text{pounds} \times \text{inches}^2\right]$

L = Length in feet

Δ = Lateral displacement of top of pier in feet due to changes in lengths of slab span caused by temperature, shrinkage, etc.

K_{AB} = Relative $\frac{bd^3}{12}$ value and d is depth of small end of member

Constants C''_{AB}, k'_1, and k'_2 may be taken from Fig. 2.

SIGN CONVENTIONS

The rules for signs observed in this discussion are as follows:

1. Any bending moment at the end A of a member AB which tends to rotate the joint A in clockwise direction is considered positive. If it tends to rotate the joint A in a counter-clockwise direction, it is considered negative.*

2. Any joint rotation angle such as Θ_A which occurs where all the member ends at the joint A rotate in a clockwise direction from their original positions is considered positive. If the rotation is in the counter-clockwise direction it is considered negative.

3. These signs are operative only while solving for moments at the joints. It is a desirable procedure to plot these moments on the side of the axis of the member on which tensile deformations take place. When this is done the location of critical sections for steel reinforcing in reinforced concrete frame members is facilitated.

DEVELOPMENT OF RIGID FRAME BRIDGE FORMULAE

Case 1—Single Span Rigid Frame. (See Fig. 3 and 4 for notation.)

Using Formulae 1 and 3, write moment equations for members AB and Aa

$$M_{AB} = \pm M^{F'}_{AB} - K_{AB}\,[C_{AB}\Theta_A + C'_{AB}\Theta_B]$$

$$M_{Aa} = \pm M^{F'}_{Aa} - K_{Aa}\,[C''_{Aa}\Theta_A]$$

*If it is desired to use the convention where a plus sign is given to a clockwise tendency of moment to rotate member then all signs in the solution of rigid frame equations may be changed. The result will of course be the same.

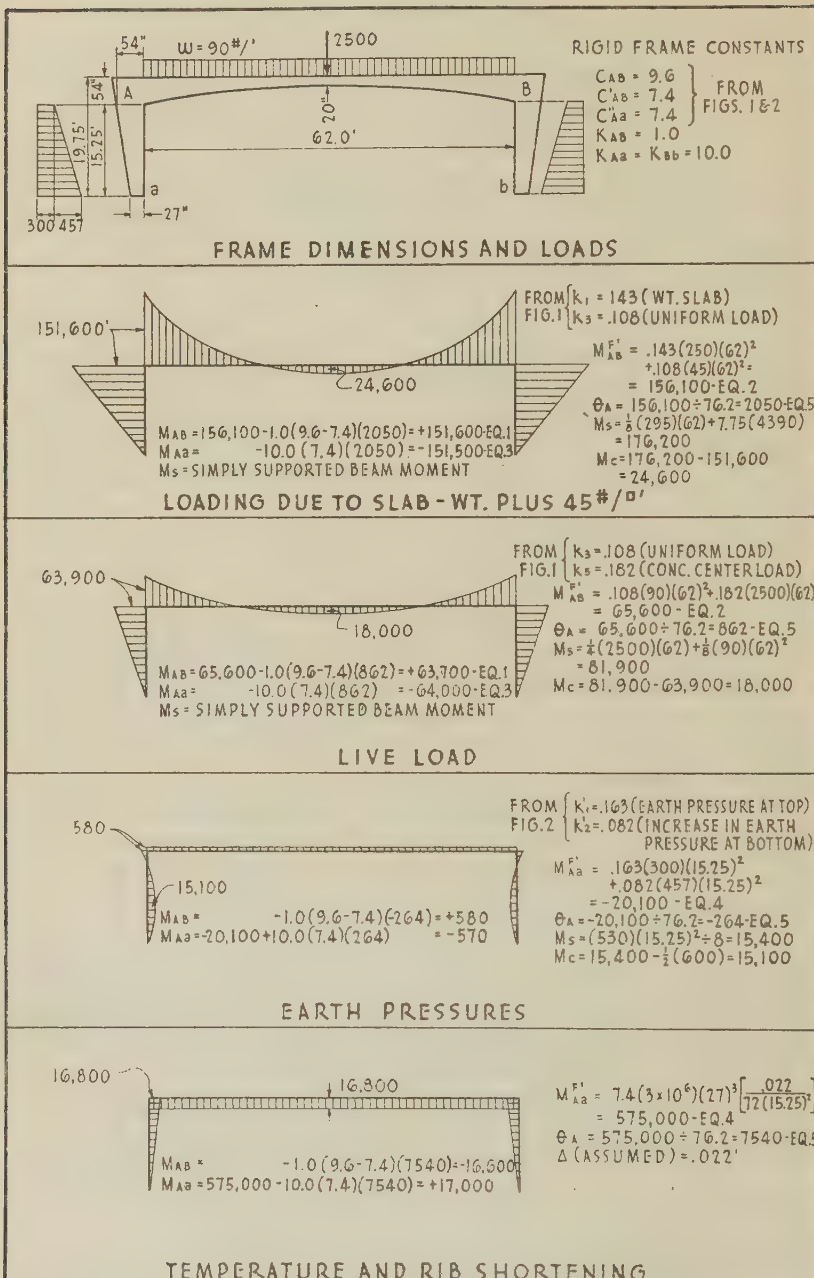


FIG. 3—SOLUTION OF SINGLE SPAN RIGID FRAME BRIDGE—CLEAR LENGTHS OF MEMBERS

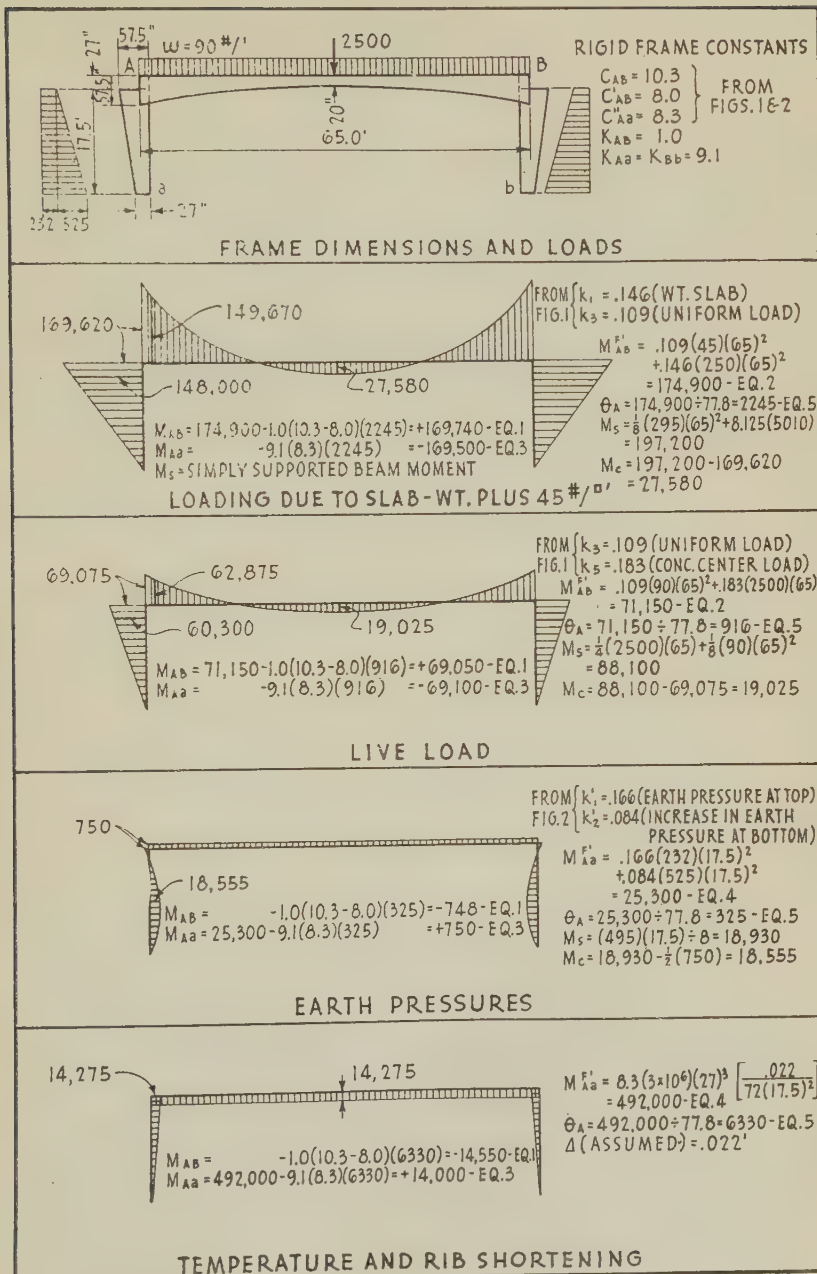


FIG 4—SOLUTION OF SINGLE SPAN RIGID FRAME BRIDGE—CENTER TO CENTER LENGTHS OF MEMBERS

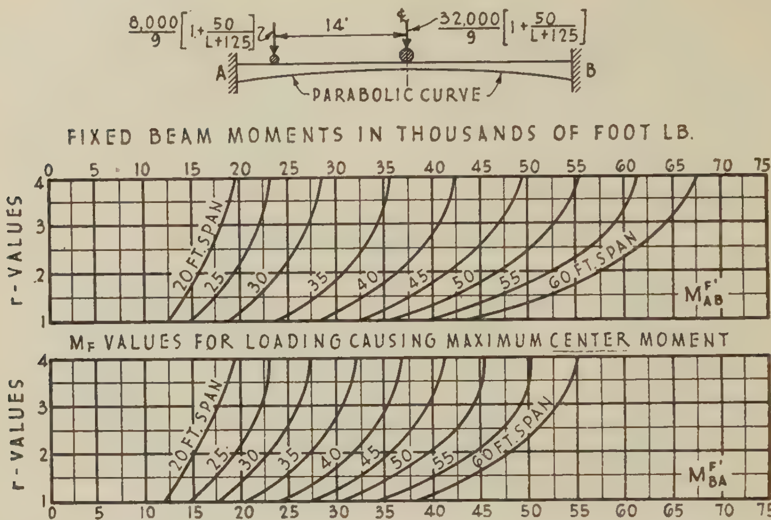


FIG. 5—FIXED END MOMENTS FOR PARABOLIC SLAB SPAN MEMBER CAUSING MAXIMUM CENTER MOMENT FOR TRUCK LOADING

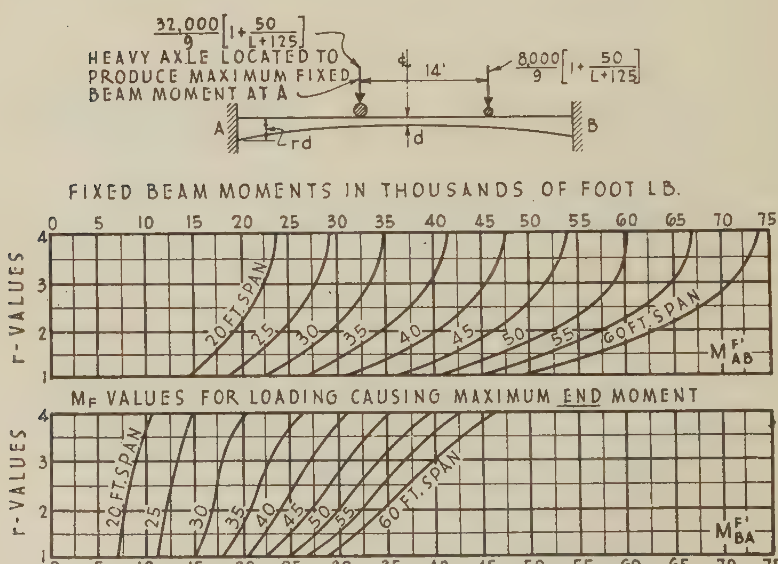


FIG. 6—FIXED END MOMENTS FOR PARABOLIC SLAB SPAN MEMBER CAUSING MAXIMUM END MOMENT FOR TRUCK LOADING

For equilibrium at the joint A the sum of the final moments around this joint must equal zero, or $M_{AB} + M_{Aa} = 0$. Therefore

$$\pm M^{F'}_{AB} \pm M^{F'}_{Aa} = K_{AB}\,[C_{AB}\Theta_A + C'_{AB}\Theta_B] + K_{Aa}\,[C''_{Aa}\Theta_A]$$

Let $\pm M^{F'}_{AB} \pm M^{F'}_{Aa} = \pm \Sigma M^{F'}_A$ = the algebraic sum of the fixed beam moments at Joint A (the unbalanced moments tending to rotate Joint A).

Let $K_{AB} = 1$, from symmetry $\Theta_A = -\Theta_B$. Then

$$\Theta_A = \pm \frac{\Sigma M^{F'}_A}{[C_{AB} - C'_{AB} + K_{Aa}C''_{Aa}]} \quad \text{.......................(5)}$$

PROCEDURE IN NUMERICAL SOLUTION

(a) Determine relative K values for different members. In case shown in Fig. 3.

$$K_{AB} = \frac{20^3}{62} \qquad K_{Aa} = \frac{27^3}{15.25} \text{ (since } b = 12 \text{ inches).}$$

Assuming $K_{AB} = 1$, relative $K_{Aa} = 10$.

(b) Obtain constants C, C' or C'' from Fig. 1 and 2 using the proper r value. In this case r for slab span member $= \frac{54}{20} = 2.7$. r for leg member $= \frac{54}{27} = 2.0$.

(c) Calculate value of $M^{F'}$. This is done by taking proper k coefficients from Fig. 1 and 2. Using these k coefficients, calculate the "fixed beam" moments as is indicated in Equation (2) and Equation (4). It is here noted that the effect of rib shortening (secondary stress case) may be treated like other loadings where the value of $\frac{[C''_{Aa}EI_a\Delta]}{72L^2}$ is the fixed beam moment induced at the top end of the post when the bottom is hinged and the axis of the post is rotated through an angle whose tangent or whose radian is $\frac{\Delta}{L}$.

(d) All moments calculated in foot-pounds. Complete numerical solution of single span rigid frame bridges is shown in Fig. 3 and 4.

Case 1a—Single Span Rigid Frame Bridge Truck Loading.

Some specifications require that bridges with spans of 60 ft. or less shall be designed for a truck loading with axles spaced on 14 ft. centers. Fig. 5 and 6 will be found very useful in such cases. Fig. 5 is so constructed that, for given values of r, the "fixed beam moments" $M^{F'}_{AB}$ and $M^{F'}_{BA}$ may be read directly in thousands of foot-pounds, the load being in position to give maximum center moment.

Fig. 6 gives the fixed beam moments $M^{F'}_{AB}$ and $M^{F'}_{BA}$ directly, for given values of r, in thousands of foot-pounds, the loads being in position to give maximum end moments.

The diagrams have been drawn for an H20 A.A.S.H.O. truck having 80 per cent of its load on the rear axle. To this has been added the specified impact $\left[1 + \frac{50}{L + 125}\right]$ and the load has been distributed over a 9-ft. lane. H15 loading may be computed

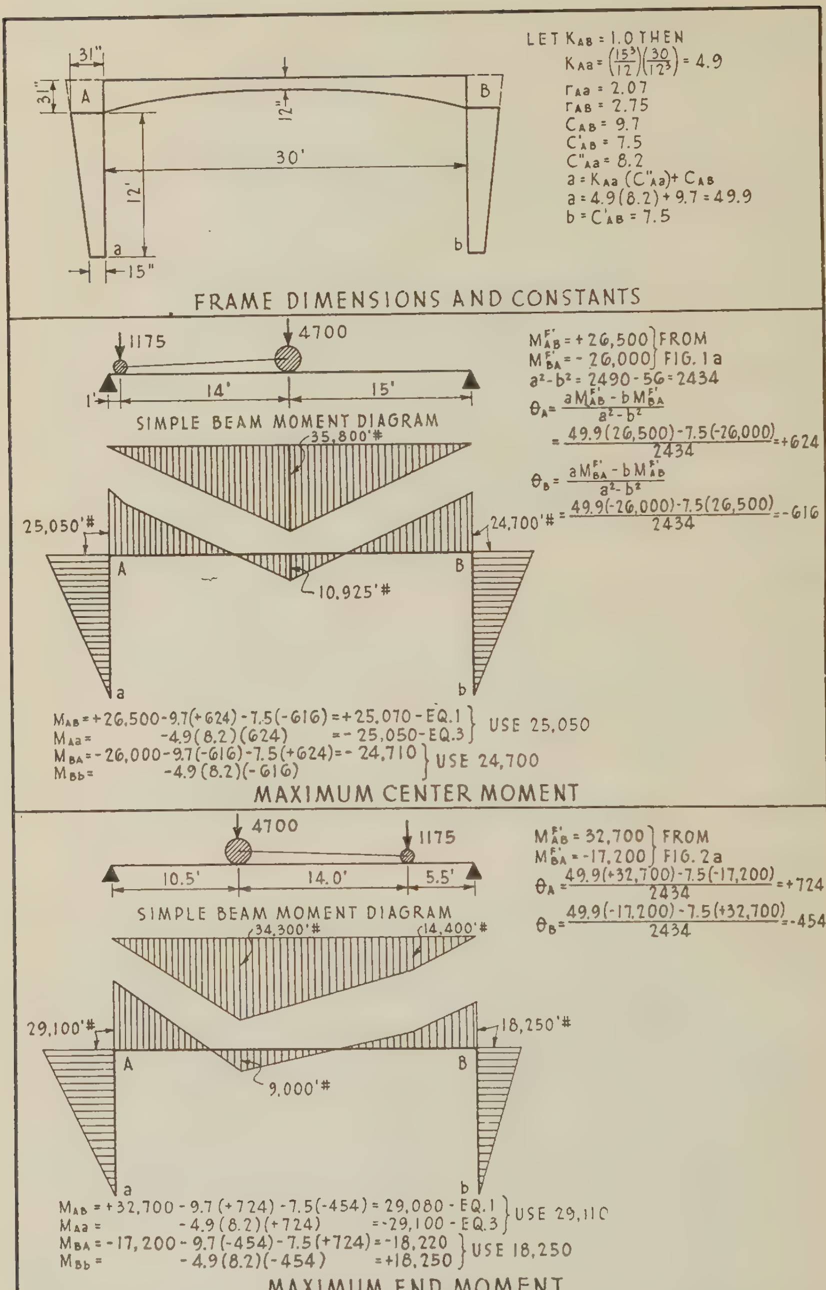
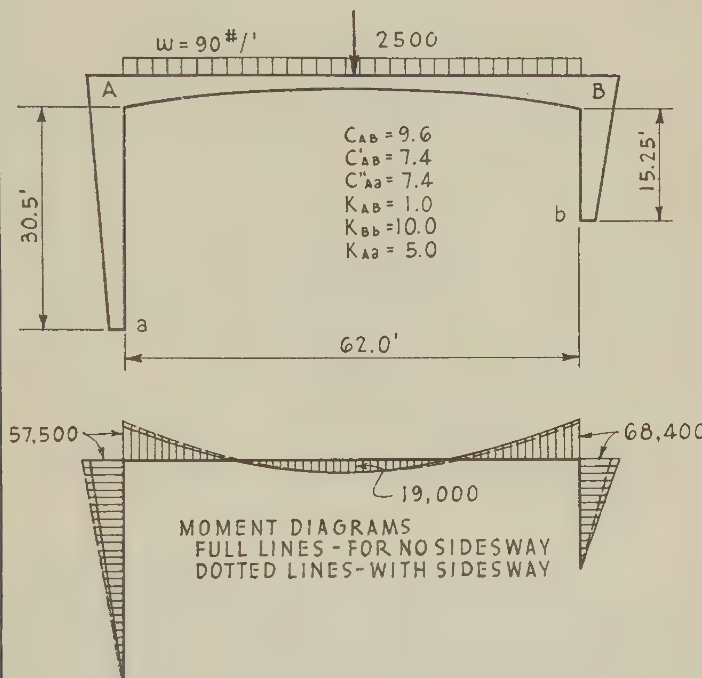


FIG. 7—SOLUTION OF SINGLE SPAN FOR TRUCK LOADING



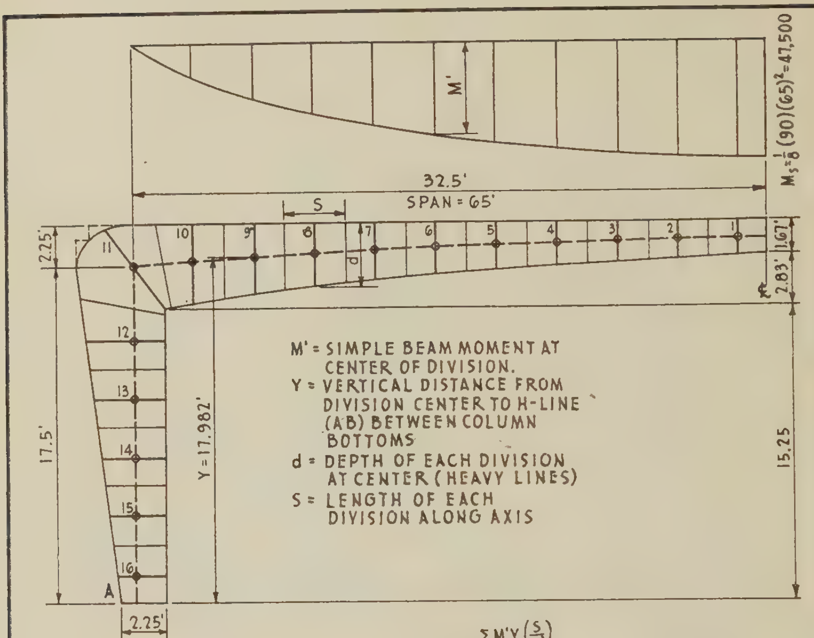
SOLUTION - NO SIDESWAY

$$0 = \begin{cases} M_{AB} = +65{,}600 - 1[9.6\,\theta_A + 7.4\,\theta_B] = +65{,}600 - [9.6(+1555) + 7.4(-923)] \\ \quad = 57{,}530 \\ M_{Aa} = \qquad -5[7.4\,\theta_A] \qquad = \qquad -5(7.4)(+1555) \\ \quad = -57{,}500 \end{cases}$$

$$0 = \begin{cases} M_{BA} = -65{,}600 - 1[9.6\,\theta_B + 7.4\,\theta_A] = -65{,}600 - [9.6(-923) + 7.4(+1555)] \\ \quad = 68{,}550 \\ M_{Bb} = \qquad -10[7.4\,\theta_B] \qquad = \qquad -10(7.4)(-923) \\ \quad = -68{,}300 \end{cases}$$

JOINT A: $46.6\,\theta_A + 7.4\,\theta_B = +65{,}600$
JOINT B: $7.4\,\theta_A + 83.6\,\theta_B = -65{,}600$
SOLVING: $\theta_B = -923$
AND $\theta_A = +1555$

FIG. 8—SOLUTION OF UNSYMMETRICAL FRAME FOR LIVE LOAD



$$H = \frac{\Sigma M'Y\left(\frac{S}{d^3}\right)}{\Sigma Y^2\left(\frac{S}{d^3}\right)} \qquad M_{AB} = 17.5\,H$$

DIVISION	M'	Y	d	S	S/d^3
1	47,300	18.914	1.681	3.1	.6530
2	46,500	18.890	1.732	3.1	.5960
3	44,800	18.839	1.848	3.1	.4900
4	42,200	18.761	2.024	3.1	.3740
5	38,700	18.657	2.247	3.1	.2730
6	34,600	18.532	2.525	3.1	.1920
7	29,200	18.374	2.868	3.1	.1410
8	23,200	18.193	3.262	3.1	.0896
9	16,100	17.982	3.712	3.1	.0604
10	8,160	17.744	4.230	3.1	.0412
11	0	17.500	5.400	3.7	.0235
12	0	13.725	4.275	3.05	.0392
13	0	10.675	3.825	3.05	.0545
14	0	7.625	3.375	3.05	.0795
15	0	4.575	2.925	3.05	.1222
16	0	1.525	2.475	3.05	.2010

RISE AT CENTER = 18.92'			
Y	Y^2	$M'Y(S/d^3)$	$Y^2(S/d^3)$
18.914	357.5	584,000	233.5
18.890	356.5	524,000	212.5
18.839	355.0	413,500	174.0
18.761	352.0	296,000	131.5
18.657	348.0	197,200	95.2
18.532	344.0	123,200	66.0
18.374	342.0	75,600	48.2
18.193	331.0	37,800	29.7
17.982	323.5	17,500	19.5
17.744	315.0	5,950	13.0
17.500	306.0		7.2
13.725	188.0		7.4
10.675	114.0		6.2
7.625	58.2		4.6
4.575	20.9		2.6
1.525	2.3		.5
		$\Sigma = 2,274,750$	$\Sigma = 1,051.0$

FOR TEMPERATURE AND RIB SHORTENING

$\Delta = H\,\Sigma y^2 \frac{ds}{EI} = .022$

$\Sigma y^2 \frac{ds}{EI} = \Sigma y^2 \frac{S}{d^3}\left[\frac{1}{\frac{1}{12}E}\right]$

$= \frac{1051 \times 12}{432,000,000} = \frac{1}{34,100}$

$\therefore H = .022 \times 34,200 = 753$

$M_A = 753 \times 17.50 = 13,200'\#$

FOR NORMAL CASE OF HORIZONTAL EXTRADOS (RISE = 18.92)

$H = \frac{2,274,750}{1,051.0} = 2,164$

FIG. 9—SOLUTION OF SINGLE SPAN BY TWO-HINGED ARCH METHOD

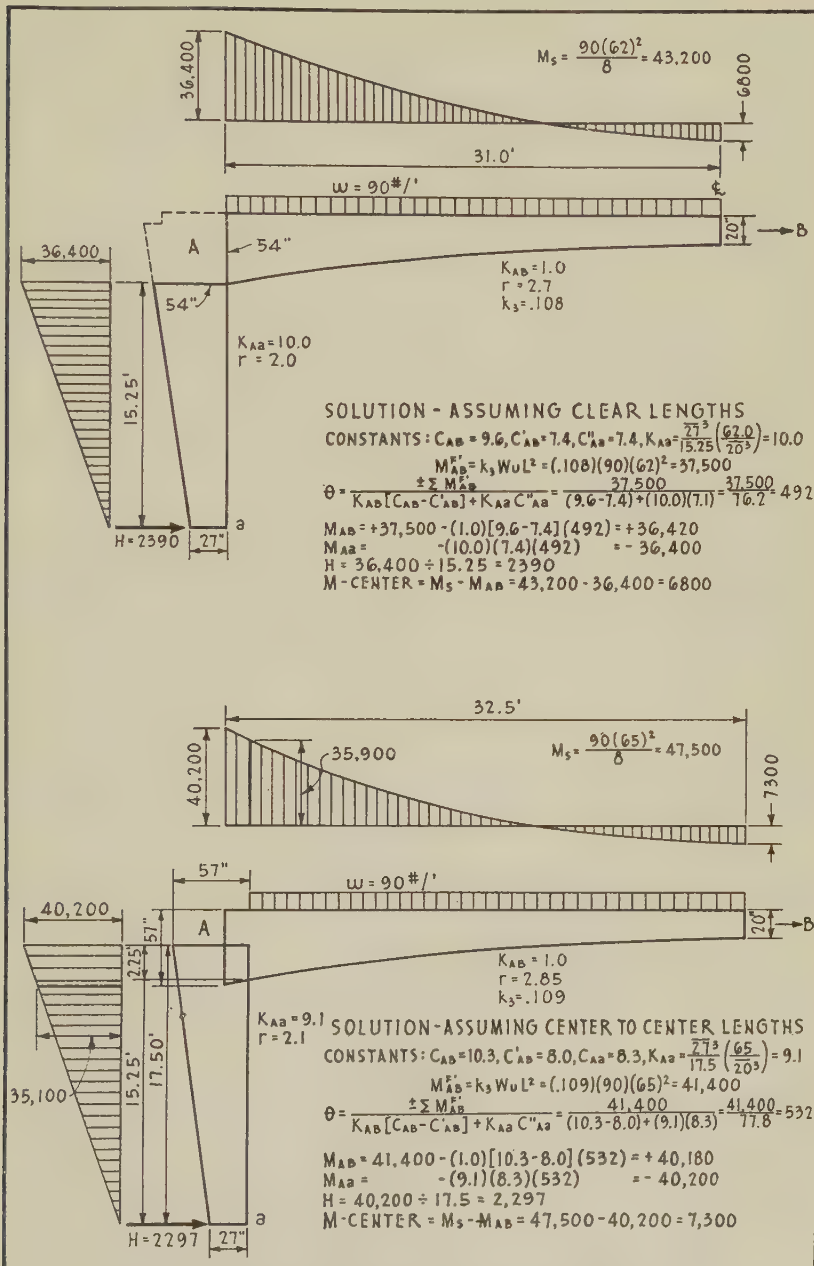


FIG. 10—COMPARISON OF COMPUTED MOMENTS USING CLEAR SPAN AND CENTER TO CENTER LENGTH

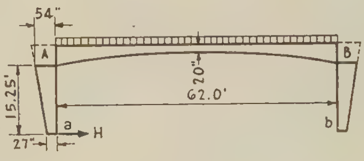
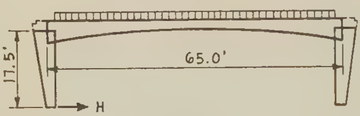
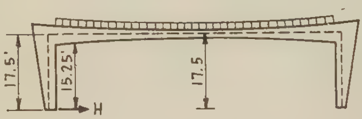
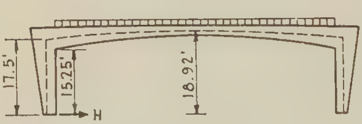
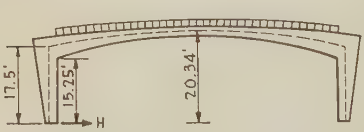
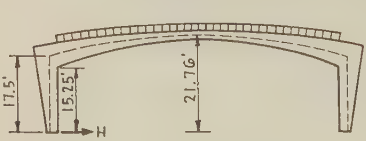
CASE (UNIFORM LOADING OF 90# PER SQ.FT.)	METHOD OF ANALYSIS	THRUST (H)	MOMENT AT CENTER OF SPAN	MOMENT AT TOP OF CLEAR POST HT. (15.25' UP)	MOMENT AT EDGE OF CLEAR SLAB SPAN
 (A)	RIGID FRAME [MEMBERS Aa & AB] CLEAR SPANS USED	2390#	6800'#	36,400'#	36,400'#
 (B)	SAME AS ABOVE EXCEPT CENTER TO CENTER SPANS USED	2297# ↑	7300'# ↑	35,100'# ↑	35,900'# ↑
		CHECK METHODS OF RIGID FRAME WITH ARCH ANALYSIS			
 (C)	STANDARD 2 HINGED ARCH ANALYSIS HORIZONTAL MEMBER STRAIGHT (16 DIVISIONS) NO CAMBER	↓ 2303#	↓ 7200'#	↓ 35,500'#	↓ 36,240'#
 (D)	SAME AS ABOVE EXCEPT RISE AT CENTER= 18.92' CAMBER=1.42	2164#	6557'#	33,000'#	31,910'#
 (E)	SAME AS ABOVE EXCEPT RISE AT CENTER= 20.34' CAMBER=2.84	2035#	6108'#	31,000'#	31,910'#
 (F)	SAME AS ABOVE EXCEPT RISE AT CENTER= 21.76' CAMBER=4.26	1926#	5590'#	29,400'#	30,260'#

FIG. 11—COMPARISON OF RESULTS OF RIGID FRAME AND ARCH ANALYSIS FOR VARIABLE CURVATURES IN SLAB AXIS

by taking $\frac{3}{4}$ of values as given, or if different lane widths are used the loading will be in direct proportion.

If, in the case of a symmetrical pin ended frame with parabolic slab span, we apply Equations (1) and (3) to joints A and B (Θ_A does not equal Θ_B due to dissemmetry of load) we find

$$\Theta_A = \frac{a\ M^{F'}{}_{AB} - b\ M^{F'}{}_{BA}}{a^2 - b^2} \text{ and}$$

$$\Theta_B = \frac{a\ M^{F'}{}_{BA} - b\ M^{F'}{}_{AB}}{a^2 - b^2} \text{ where}$$

$$a = K_{Aa}\ C''_{Aa} + C_{AB} \text{ and } b = C'_{AB}$$

When using these equations, K_{AB} (relative I/L of parabolic span member) must be computed at center of span and made equal to unity.

A complete numerical solution for truck load is shown in Fig. 7. Dead load, earth pressure, temperature and shrinkage would be computed as shown in Fig. 3.

UNSYMMETRICAL SINGLE SPAN

The method of solution to be followed where abutments are of different height is shown in Fig. 8. Since Θ_A does not equal Θ_B in this case, Equation (5) does not hold. The numerical solution shown is for the case of uniform live load with single concentration at center for no side sway. Moments for other types of loading would follow the same general procedure. Side sway was computed for this case and is indicated to scale by dotted line on the figure. Note that the positive moment is unchanged. A comparison with Fig. 3 will show the end moment at A is somewhat less, and at B slightly larger.

EFFECT OF CAMBER AND CHOICE OF SPAN LENGTH

In Fig. 11, in tabular form, are given the results of an analysis of six different frame conditions. This study is given to illustrate the effect of the assumption as to span length and the effect of camber.

Uniform loading of 90 lb. per sq. ft. is used in all cases. Uniform loading was chosen as it gives a moment distribution between that of concentrated center load and dead load increasing toward ends of slab member.

Cases A and B were computed by methods of this paper and are shown in Fig. 10. The calculations in Cases C to F were made by the standard two-hinged arch method, using 16 divisions, as shown in Fig. 9. In two-hinged analysis full effect of camber is considered. In the method of this paper the axes of members are treated as straight lines. By comparing Cases A and B with Case D it is seen that Case A, using clear span, gives the critical center moment more accurately than does Case B using center to center span. Case B gives a better value for the less sensitive moment at top of post or at edge of clear span. All values of Cases A and B are on the side of safety. For design purposes, the moment at the face of abutment is required, and since the clear span solution gives this directly without interpolation, it is preferred.

Design on the basis of side sway prevented is on the side of safety since the larger end moment would be slightly decreased if side sway were considered. Arching of the slab axis tends to decrease all moments. The reduction due to arching will be greater than the increase due to side sway. It is believed that neglect of these two effects is unimportant and that such refinements are not justified.

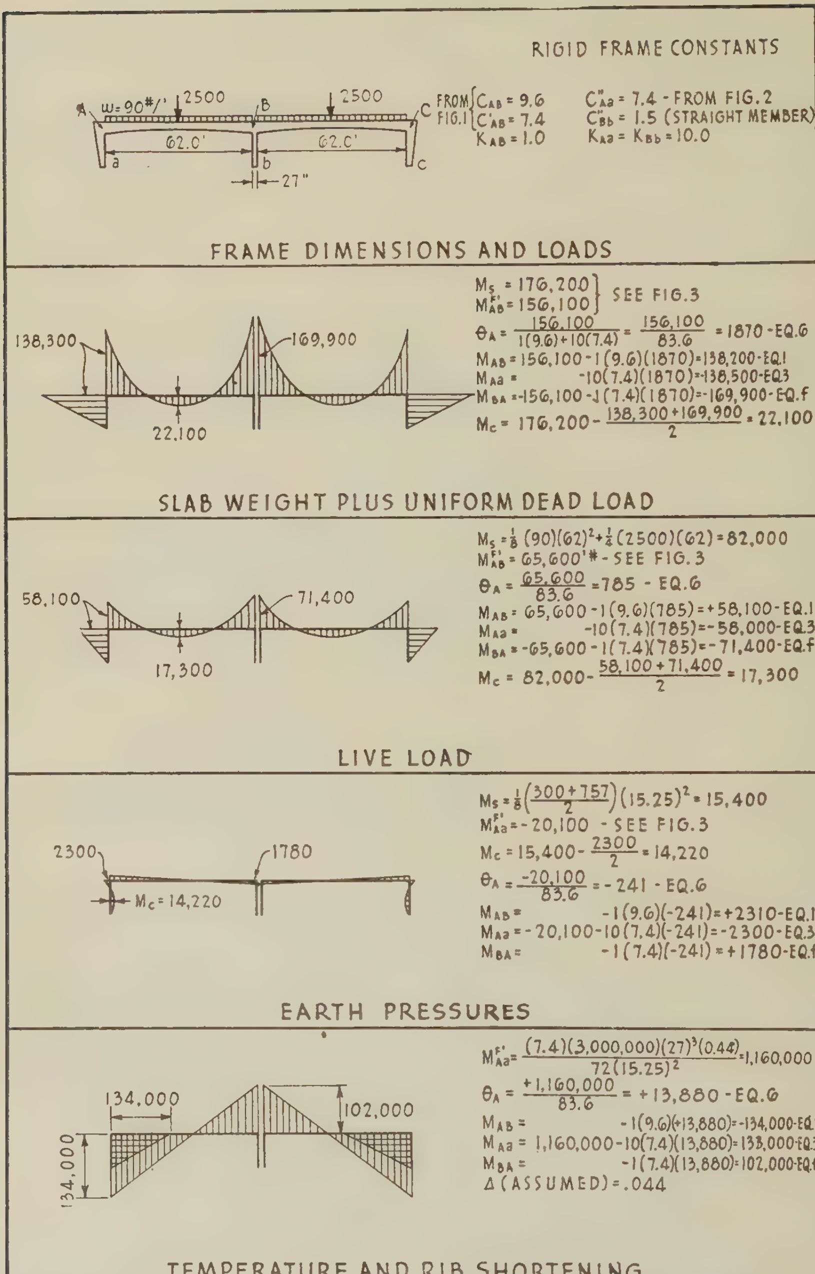
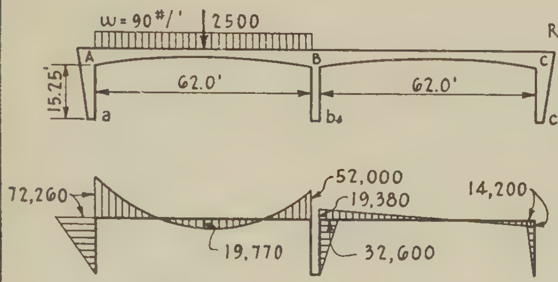


FIG. 12—TWO-SPAN SYMMETRICAL CASE—SYMMETRICAL LOADING



RIGID FRAME CONSTANTS

$C_{AB} = 9.6$, $C'_{AB} = 7.4$, $C''_{Aa} = 7.4$ — FROM FIGS 1&2
$C''_{Bb} = 1.5$
$K_{Aa} = 10.0 = K_{Bb} = K_{Cc}$
$K_{AB} = 1.0 = K_{BC}$
$M^{F'}_{AB} = +65,600$, $M^{F'}_{BA} = -65,600$ — SEE FIG. 3

$$0 = \begin{cases} M_{AB} = +65,600 - 9.6\,\theta_A - 7.4\,\theta_B = +65,600 - 9.6(976) - 7.4(-2170) = +72,320 \text{ - EQ.1} \\ M_{Aa} = -10.0(7.4)\theta_A = (-10.0)(7.4)(976) = -72,200 \text{ - EQ.3} \end{cases}$$

$$0 = \begin{cases} M_{BA} = -65,600 - 9.6\,\theta_B - 7.4\,\theta_A = -65,600 - 9.6(-2170) - 7.4(976) = -52,000 \\ M_{BC} = -9.6\,\theta_B - 7.4\,\theta_C = -9.6(-2170) - 7.4(192) = +19,380 \\ M_{Bb} = -10.0(1.5)\theta_B = -10.0(1.5)(-2170) = +32,600 \end{cases}$$

$$0 = \begin{cases} M_{CB} = -9.6\,\theta_C - 7.4\,\theta_B = -9.6(192) - 7.4(-2170) = +14,200 \\ M_{Cc} = -10.0(7.4)\,\theta_C = -10.0(7.4)(192) = -14,200 \end{cases}$$

FOR JOINT A: $83.6\,\theta_A + 7.4\,\theta_B = +65,600$
FOR JOINT B: $7.4\,\theta_A + 34.2\,\theta_B + 7.4\,\theta_C = -65,600$
FOR JOINT C: $7.4\,\theta_B + 83.6\,\theta_C = 0$

SOLUTION OF EQUATIONS

	LEFT HAND MEMBER			RIGHT HAND MEMBER
JOINT	θ_A	θ_B	θ_C	
A	83.6	7.4		+65,600
B	7.4	34.20	7.4	-65,600
C		7.4	83.6	0.0
	-7.4	-.65		- 5,810
		33.55	7.4	-71,410
		-33.55	-379.0	
			-371.6	-71,410

$\theta_C = +192$

$\theta_B = -\frac{83.6}{7.4}(192) = -2170$

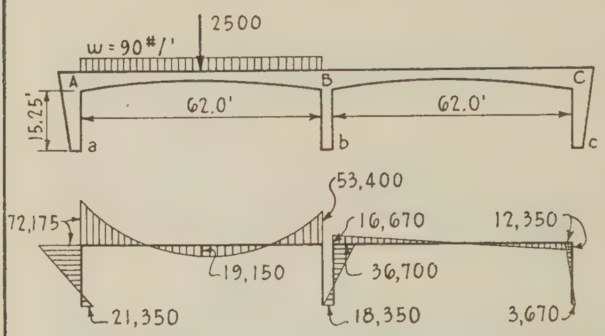
$\theta_A = \frac{65.6 - 7.4(-2170)}{83.6} = +976$

APPROX-IMATION	θ_A	θ_B	θ_C	
	83.6	7.4		+65,600
	7.4	34.2	7.4	-65,600
		7.4	83.6	
1ST	+785	-1920	0	
2ND	+955	-2160	+170	
3RD	+976	-2160	+191	

PROCEDURE FOR CONVERGING APPROXIMATION

1ST: $\frac{65,600}{83.6} = \theta_A$, AND $\frac{-65,600}{34.2} = \theta_B$
2ND: FIND θ_C USING 1ST VALUES OF θ_B
FIND θ_A USING 1ST VALUES OF θ_B
FIND θ_B USING 2ND VALUES OF θ_A & θ_C
3RD: SIMILAR TO SECOND (ETC.)

FIG. 13—TWO-SPAN SYMMETRICAL CASE—UNSYMMETRICAL LOADING



RIGID FRAME CONSTANTS

$C_{AB} = 9.6$
$C'_{AB} = 7.4$
$C_{Aa} = 9.8$
$C'_{Aa} = 2.9$
$C_{Bb} = 2.0$
$C'_{Bb} = 1.0$
$K_{Aa} = 10.0 = K_{Bb} = K_{Cc}$
$K_{AB} = 1.0 = K_{BC}$
$M^{F'}_{AB} = +65,600$
$M^{F'}_{BA} = -65,600$

$$0 = \begin{cases} M_{AB} = +65,600 - 9.6\theta_A - 7.4\theta_B = 65,600 - 9.6(737) - 7.4(-1385) = +72,100 \\ M_{Aa} = -10.0(9.8)\theta_A = -10.0(9.8)(737) = -72,250 \end{cases}$$

$$0 = \begin{cases} M_{BA} = -65,600 - 9.6\theta_A - 7.4\theta_A = 65,600 - 9.6(-1835) - 7.4(737) = -53,400 \\ M_{BC} = -9.6\theta_B - 7.4\theta_C = -9.6(-1835) - 7.4(126) = +16,670 \\ M_{Bb} = -10.0(2)\theta_B = -10.0(2)(-1835) = +36,700 \end{cases}$$

$$0 = \begin{cases} M_{CB} = -9.6\theta_C - 7.4\theta_B = -9.6(126) - 7.4(-1835) = +12,360 \\ M_{Cc} = -10.0(9.8)\theta_C = -10.0(9.8)(126) = -12,350 \end{cases}$$

$$M_{aA} = K_{Aa}C'_{Aa}\theta_A = 10.0(2.9)(737) = 21,350$$
$$M_{bB} = K_{Bb}C'_{Bb}\theta_B = 10.0(1.0)(-1835) = -18,350$$
$$M_{cC} = K_{Cc}C'_{Cc}\theta_C = 10.0(2.9)(126) = 3,670$$

FOR JOINT A: $107.6\theta_A + 7.4\theta_B = +65,600$
FOR JOINT B: $7.4\theta_A + 39.2\theta_B + 7.4\theta_C = -65,600$
FOR JOINT C: $7.4\theta_B + 107.6\theta_C = 0$

SOLUTION OF EQUATIONS

JOINT	LEFT HAND MEMBER			RIGHT HAND MEMBER
	θ_A	θ_B	θ_C	
A	107.6	7.4		+65,600
B	7.4	39.2	7.4	-65,600
C		7.4	107.6	0
	1.0	.0688		611
	1.0	5.3	1.0	-8,870
		5.231	1.0	-9,481
		1.0	.192	-1,810
		1.0	14.55	0
			14.358	+ 1,810

$\theta_C = 1810 \div 14.358 = 126$
$\theta_B = -126(14.55) = -1835$
$\theta_A = 611 - .0688(-1835) = 737$

FIG. 14—TWO-SPAN SYMMETRICAL CASE—UNSYMMETRICAL LOADING (BOTTOMS OF POSTS FIXED)

Case 2—Two-Span Symmetrical Rigid Frame Bridge. (See Fig. 12 for notation.)

Using Formulae (1) and (3), write moment equations for members AB and Aa.

$$M_{AB} = \pm M^{F'}_{AB} - K_{AB}\,[C_{AB}\Theta_A + C'_{AB}\Theta_B]$$
$$M_{Aa} = \pm M^{F'}_{Aa} - K_{Aa}\,[C''_{Aa}\Theta_A]$$

For equilibrium $M_{AB} + M_{Aa} = 0$. Also, $\Theta_B = 0$ since there can be no rotation at top of pier when framing and loading are symmetrical.

Make $K_{AB} = 1$

Then $\pm M^{F'}_{AB} \pm M^{F'}_{Aa} = \pm \Sigma M^{F'}_A = C_{AB}\Theta_A + K_{Aa}C''_{Aa}\Theta_A$

$$\text{Therefore } \Theta_A = \frac{\pm \Sigma M^{F'}_A}{[C_{AB} + K_{Aa}C''_{Aa}]} \qquad (6)$$

From symmetry $\Theta_C = -\Theta_A$, $M_{AB} = -M_{CB}$, $M_{BA} = -M_{BC}$ and $M_{Bb} = 0$. Now apply Equation (1) to Joint B

$$M_{BA} = -M^{F'}_{BA} - K_{AB}\,[C'_{AB}\Theta_A] \qquad \text{(f)}$$

When Θ_A has been determined by using Equation (6), numerical solution follows same procedure as outlined for single span rigid frame bridges. Complete numerical solution is shown in Fig. 12. It will be noted in this figure that the loadings and dimensions of frame members are identical with those of the single span shown in Fig. 3 except that the center pier is a member of uniform thickness of 27 inches and $C''_{Bb} = 1.5$ as shown in Equation (e).

Case 2a—Two-Span Unsymmetrically Loaded Rigid Frame Bridge. (See Fig. 13 for notation.)

The solution for dead load, earth pressure and rib-shortening is the same as shown in Fig. 12.

The live loading giving maximum positive moment near center of span occurs when one span only is loaded with live load.

Solution of this case involves three unknown Θ values for pin ended condition since side sway is assumed to be prevented.

The analysis is best made by applying Equations (1) and (3) to Joints A, B and C in succession using numerical quantities.

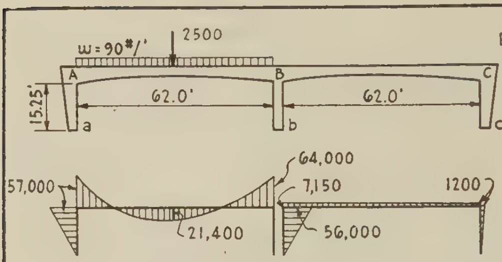
The three simultaneous equations have been solved by two methods. When the number of unknowns does not exceed four, the long hand method is as easy as any. The "converging approximations" method is very useful where the number of unknowns becomes large.

For comparative purposes and to illustrate the method of solution, a two-span unsymmetrical live load case with bottom of posts fixed has been solved and is shown in Fig. 14.

Case 2b—Two-Span Unsymmetrically Loaded Rigid Frame Bridge Side Sway Considered. (See Fig. 15 for notation.)

To illustrate the method of attack and to give a comparison of results that may be expected, this problem has been worked out in Fig. 15 for the case of live load on one span only, assuming pin ended pier members.

$M^{F'}_{Aa}$ may be computed as in previous problems and C''_{Aa} may be found in Fig. 2.



RIGID FRAME CONSTANTS

$C_{AB} = 9.6$, $C'_{AB} = 7.4$, $C''_{Aa} = 7.4$, $C''_{Bb} = 1.5$ — FROM FIGS. 1 & 2

$K_{Aa} = 10.0 = K_{Bb} = K_{Cc}$

$K_{AB} = 1.0 = K_{BC}$

$M^F_{AB} = +65,600$, $M^F_{BA} = -65,600$ — SEE FIG. 3

$$0 = \begin{cases} M_{AB} = +65{,}600 - 9.6(\theta_A) - 7.4(\theta_B) = +65{,}600 - 9.6(+2502) - 7.4(-2064) = +56{,}900 \text{ - EQ. i} \\ M_{Aa} = -10.0(7.4)(\theta_A - R) = -10.0(7.4)(+2502 - 1732) = -57{,}000 \text{ - EQ. j} \end{cases}$$

$$0 = \begin{cases} M_{BA} = -65{,}600 - 9.6(\theta_B) - 7.4(\theta_A) = -65{,}600 - 9.6(-2064) - 7.4(+2502) = -64{,}300 \\ M_{BC} = -9.6(\theta_B) - 7.4(\theta_C) = -9.6(-2064) - 7.4(+1717) = +7{,}150 \\ M_{Bb} = -10.0(1.5)(\theta_B - R) = -10.0(1.5)(-2064 - 1732) = +57{,}000 \end{cases}$$

$$0 = \begin{cases} M_{CB} = -9.6(\theta_C) - 7.4(\theta_B) = -9.6(1717) - 7.4(-2064) = -1{,}200 \\ M_{Cc} = -10.0(7.4)(\theta_C - R) = -10.0(7.4)(1717 - 1732) = +1{,}120 \end{cases}$$

$$\Sigma H = 0 \begin{cases} M_{Aa} = -10.0(7.4)(\theta_A - R) = -10.0(7.4)(+2502 - 1732) = -57{,}000 \\ M_{Bb} = -10.0(1.5)(\theta_B - R) = -10.0(1.5)(-2064 - 1732) = +57{,}000 \\ M_{Cc} = -10.0(7.4)(\theta_C - R) = -10.0(7.4)(1717 - 1732) = +1{,}120 \end{cases}$$

FOR JOINT A: $83.6\theta_A + 7.4\theta_B - 74.0R = +65{,}600$

FOR JOINT B: $7.4\theta_A + 34.2\theta_B + 7.4\theta_C - 15.0R = -65{,}600$

FOR JOINT C: $7.4\theta_B + 83.6\theta_C - 74.0R = 0$

BENT EQ. − $\Sigma H = 0$: $74.0\theta_A + 15.0\theta_B + 74.0\theta_C - 163.0R = 0$

SOLUTION OF EQUATIONS

JOINT	LEFT HAND MEMBER θ_A	θ_B	θ_C	R	RIGHT HAND MEMBER
A	83.6	7.4		−74.0	+65,600
B	7.4	34.2	7.4	−15.0	−65,600
C		7.4	83.6	−74.0	0
ΣH=0	74.0	15.0	74.0	−163.0	0
	1.0	.089		−.885	+785.0
	1.0	4.62	1.0	−2.03	−8860.0
	1.0	.203	1.0	−2.20	0
		−4.53	−1.0	+1.145	+9645
		4.42	0.0	+.17	−8860
		−1.0	−.22	+.253	+2125
		1.0	0.0	.038	−2000
		1.0	11.3	−10.00	0
			−.22	+.291	+125
			−11.3	+10.04	−2000
			−1.0	+1.32	+568
			−1.0	+0.89	−177
				.43	+745

$\therefore R = +1732$

$\theta_C = .89R = 177$
$\therefore \theta_C = 177 + (.89)(+1732) = +1717$

$\theta_B + .22\theta_C - .253R = -2000$
$\therefore \theta_B = -2000 - 374 + 438 = -2064$

$\theta_A + .089\theta_B - .885R = +785$
$\therefore \theta_A = +785 + (.089)(+1732) - (.089)(-2064)$
$\theta_A = +785 + 1534 + 183 = +2502$

FIG. 15—TWO-SPAN SYMMETRICAL CASE—UNSYMMETRICAL LOADING (SIDE SWAY CONSIDERED)

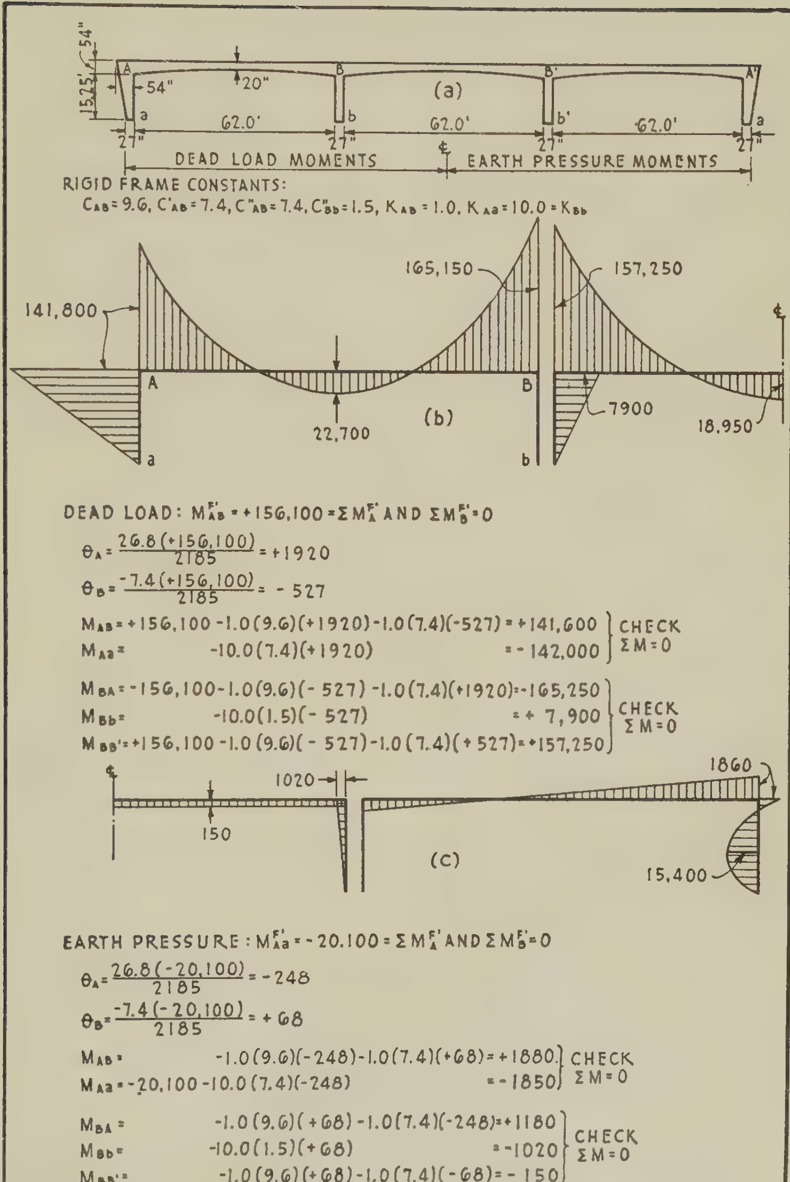


FIG. 16—SOLUTION OF THREE SPAN SYMMETRICAL FRAME DEAD LOAD AND EARTH PRESSURE

In the problem solved in Fig. 15, Equations (i) and (j)* have been applied successively to Joints A, B and C. This gives three equations and four unknowns. Since the sum of the horizontal forces $\Sigma H = O$, we can write another equation by cutting the tops of the piers at A, B and C. Then

$$\frac{M_{Aa} + 0}{h} + \frac{M_{Bb} + 0}{h} + \frac{M_{Cc} + 0}{h} = H = 0 \quad \text{or}$$

$$M_{Aa} + M_{Bb} + M_{Cc} = 0$$

These equations are solved in tabular form for Θ_A, Θ_B, Θ_C and R and these values are substituted in the original equations for the final end moments as shown.

Case 3—Three Span Symmetrically Loaded Rigid Frame Bridge—Pin Ended. (See Fig. 16 for notation.)

In this case there are two unknown Θ values or joint rotations since $\Theta_A = -\Theta'_A$ and $\Theta_B = -\Theta'_B$.

First apply equations (1) and (3) to Joint A and equate their sum to 0.

$$M_{AB} = \pm M^{F'}{}_{AB} - K_{AB}\,[C_{AB}\Theta_A + C'_{AB}\Theta_B]$$

$$M_{Aa} = \pm M^{F'}{}_{Aa} - K_{Aa}\,[C''_{Aa}\Theta_A] \quad \text{make } K_{AB} = 1$$

then $\pm M^{F'}{}_{AB} \pm M^{F'}{}_{Aa} = \Sigma M^{F'}{}_A =$ unbalanced moment at A =

$$[C_{AB} + K_{Aa}C''_{Aa}]\,\Theta_A + C'_{AB}\Theta_B \quad\text{.......... (l)}$$

Next apply equations (1) and (3) to joint B and equate their sum to 0, make $K_{AB} = 1$

$$M_{BA} = \pm M^{F'}{}_{BA} - [C_{AB}\Theta_B + C'_{AB}\Theta_A]$$

$$M_{BB'} = \pm M^{F}{}_{BB'} - [C_{AB}\Theta_B + C'_{BB'}\Theta_{B'}]$$

$$M_{Bb} = -K_{Bb}\,[C''_{Bb}\Theta_B] \qquad \Theta_B = -\Theta_{B'} \qquad C'_{BB'} = C'_{AB}$$

Unbalanced moment at $B = \pm \Sigma M^{F'}{}_B = \pm M^{F'}{}_{BA} \pm M^{F'}{}_{BB'} =$

$$[2\,C_{AB} - C'_{AB} + K_{Bb}C''B_b]\,\Theta_B + C'_{AB}\Theta_A \quad\text{.......... (m)}$$

Substitute values of constants from Figs. (1) and (2) in Equations (l) and (m) and we have

$$[9.6 + 10.0\ (7.4)\ ]\,\Theta_A + 7.4\,\Theta_B = \pm \Sigma M^{F'}{}_A$$

$$[2\ (9.6) - 7.4 + 10.0\ (1.5)\ ]\,\Theta_B + 7.4\,\Theta_A = \pm \Sigma M^{F'}{}_B$$

Solving for Θ_A and Θ_B

$$\Theta_A = 26.8\,\frac{(\pm \Sigma M^{F'}{}_A) - 7.4\,(\pm \Sigma M^{F'}{}_B)}{2185} \quad\text{.......... (n)}$$

$$\Theta_B = 83.6\,\frac{(\pm \Sigma M^{F'}{}_B) - 7.4\,(\pm \Sigma M^{F'}{}_A)}{2185} \quad\text{.......... (o)}$$

---

*From Equation (d) with notation suitable to member Aa

$$M_{Aa} = \pm M^{F}{}_{Aa} - K_{Aa}\,[C_{Aa}\Theta_A + C'_{Aa}\Theta_a - (C_{Aa} + C'_{Aa})\,R] \quad\text{.......... (g)}$$

Applying this equation to member Aa at the pin end, remembering that $M_{aA} = 0$

$$M_{aA} = 0 = -M^{F}{}_{aA} + K_{Aa}\,[\,(C_{Aa} + C'_{Aa})\,R - C_{Aa}\Theta_a - C'_{Aa}\Theta_A] \quad\text{.......... (h)}$$

$$\text{Solving for } \Theta_a = -\frac{M^{F}{}_{aA}}{K_{Aa}C_{Aa}} + \left(\frac{C_{Aa} + C'_{Aa}}{C_{Aa}}\right) R - \frac{C'_{Aa}}{C_{Aa}}\,\Theta_A \quad\text{.......... (i)}$$

Substituting Θ_a in equation (g) and simplifying gives

$$M_{Aa} = M^{F'}{}_{Aa} + K_{Aa}\,C''_{Aa}\,[R - \Theta_A] \quad\text{.......... (j)}$$

$$\text{where } M^{F'}{}_{Aa} = M^{F}{}_{aA}\left(\frac{C'_{Aa}}{C_{Aa}}\right) + M^{F}{}_{Aa} \text{ and } C''_{Aa} = C_{Aa} - \frac{(C'_{Aa})^2}{C_{Aa}} \quad\text{.......... (k)}$$

In Equations (g), (h) and (k) the values of $M^{F}{}_{aA}$ and $M^{F}{}_{Aa}$ are the common case of the fixed beam moment where both ends of the members are considered fixed but of variable moment of inertia and must not be confused with the fixed beam moment for members of constant moment of inertia. $M^{F'}{}_{Aa}$ as used in these equations is the special case used in this paper in Equation (3) and defined in the footnote to that equation.

It should be noted that for the three-span bridge all cases of live load are symmetrical to produce the worst condition of center moment.

PROPOSED METHOD OF ELIMINATION OF HIGH MOMENTS DUE TO RIB SHORTENING MULTIPLE SPAN RIGID FRAME BRIDGES

It will be noted from a comparison of the results of analysis shown on Fig. 3 with that of Fig. 17 that rib shortening moments increase rapidly with the increase in number of spans. Moments due to dead load, live load, and earth pressure do not vary greatly. It is therefore proposed that an economical design could be effected by the insertion of expansion joints such as shown in Fig. 19.

Such a joint will not change the dead load moments, which are the most important ones in long span structures. This is where the continuous rigid frame bridge with variable moment of inertia has its greatest advantages. Live load moments will be affected, though not seriously, and will require a different analysis from the one here proposed for the multiple span case. Earth pressure effects will remain about the same.

A convenient formula is given on Fig. 18 for the calculation of rib shortening moments when this type of joint is used to eliminate high moments by limiting the rib shortening effect to one of the unsymmetrical bents of the multiple bent bridge.

This formula indicates that the influence of increasing height of the end pier is to decrease rib shortening moments in proportion to the increase in height. If the bottoms of the piers were fixed very small changes in rib shortening moments would take place as the constant C'' would be replaced by a slightly higher value of C for this post member.

DISCUSSION OF ASSUMPTIONS MADE IN THIS ANALYSIS

In any problem of indeterminate analysis certain assumptions must be made which are debatable and the possible advantages of these should be carefully weighed. Certain assumptions are involved in the choice of frame dimensions, proportions of concrete and loadings. No attempt will be made to justify the frame dimensions used, nor will unit stresses be computed since the object of the paper was to present a method of analysis rather than to design a bridge. The assumptions which affect the analysis are:

1. I or the moment of inertia of all cross sections was assumed proportional to $\frac{bd^3}{12}$.

2. E or the modulus of elasticity was assumed to be constant throughout.

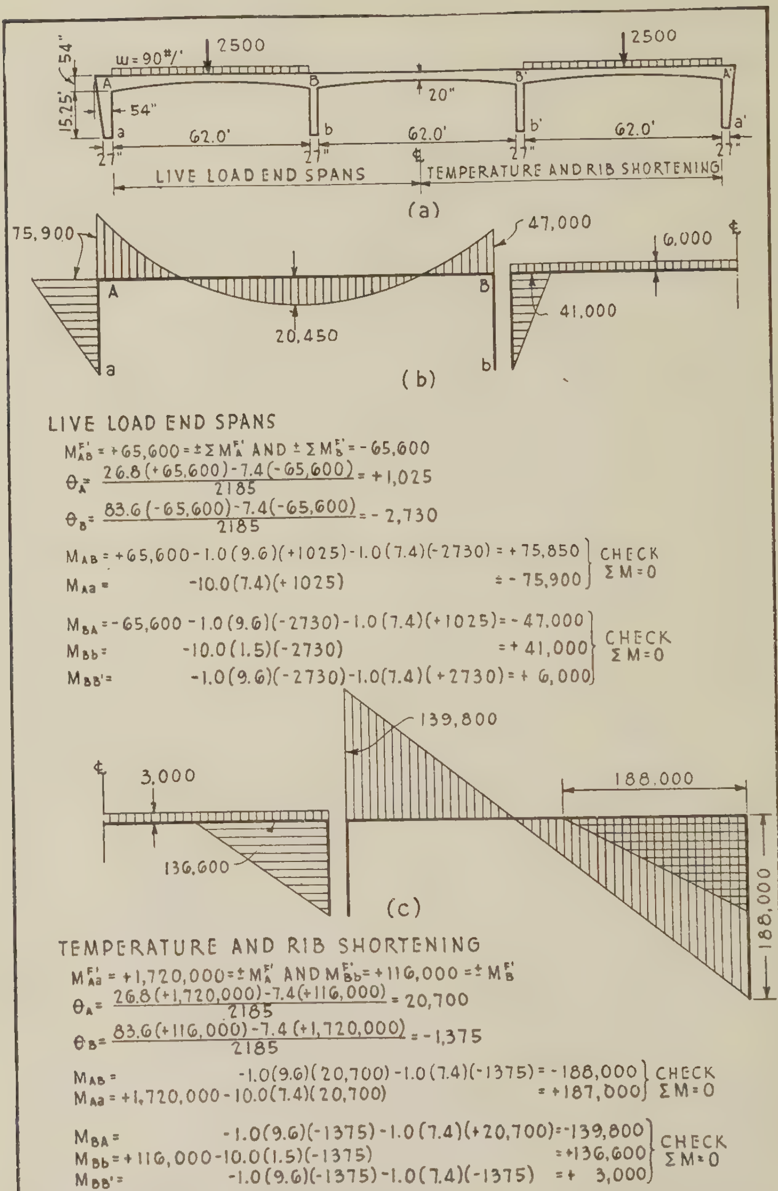
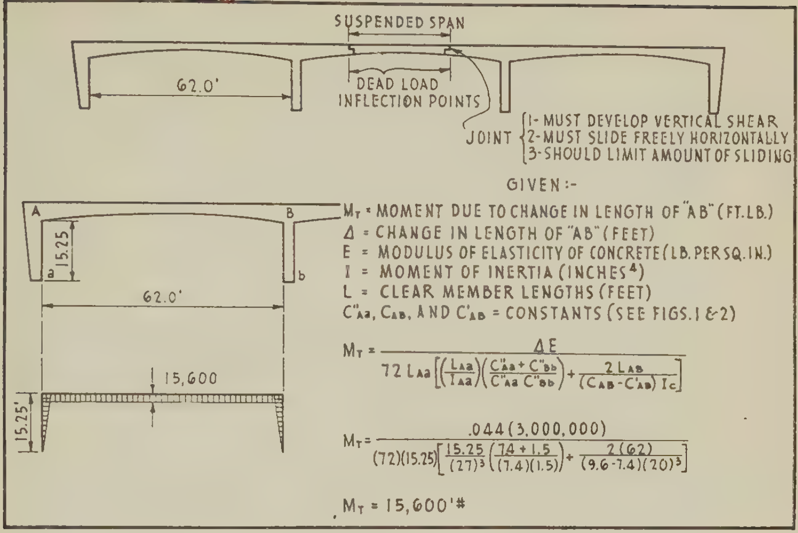


FIG. 17—SOLUTION OF THREE SPAN SYMMETRICAL FRAME LIVE LOAD AND TEMPERATURE



$$M_T = \frac{\Delta E}{72\, L_{Aa}\left[\left(\frac{L_{Aa}}{I_{Aa}}\right)\left(\frac{C''_{aa}+C''_{bb}}{C''_{aa}\,C''_{bb}}\right)+\frac{2\,L_{AB}}{(C_{AB}-C'_{AB})\,I_c}\right]}$$

$$M_T = \frac{.044(3{,}000{,}000)}{(72)(15.25)\left[\frac{15.25}{(27)^3}\left(\frac{7.4+1.5}{(7.4)(1.5)}\right)+\frac{2(62)}{(9.6-7.4)(20)^3}\right]}$$

Fig. 18—Suggestion for reducing temperature effects

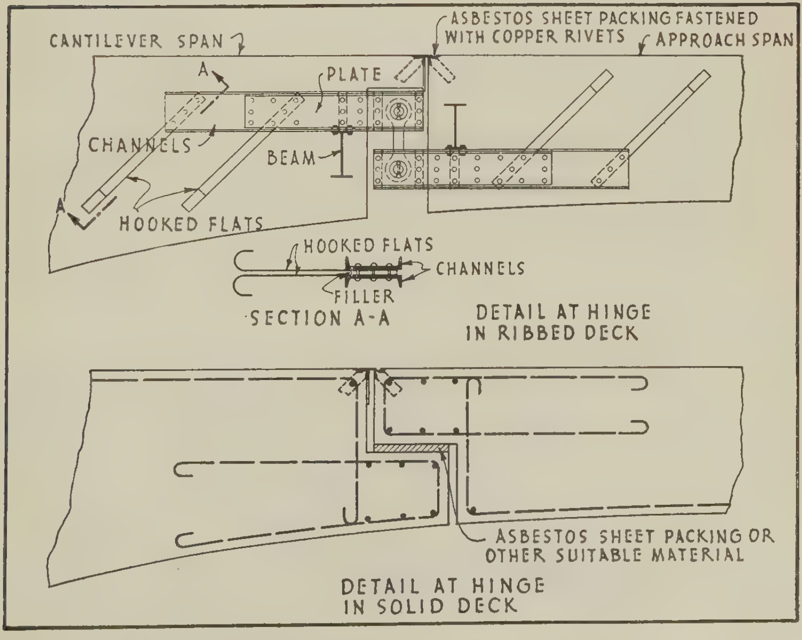


Fig. 19—Possible types of joints

3. The pier or vertical post members were taken as hinged rather than fixed at the bottom. In all probability a condition somewhere between these two actually exists. This assumption is found to affect maximum moments very little. For the rare case of very high pier members with unusually short slab spans this assumption might have to be modified and both cases of hinged and fixed bottoms would have to be investigated.

4. As previously stated, clear lengths were used instead of center-to-center lengths.

5. Side sway or effect of lateral movement of all tops of piers in the same direction with reference to all bottoms of piers resulting from unsymmetrical loading will be neglected. It is here proposed that side sway is not free to happen because of restraining active earth pressure and the passive resistance of the earth.

6. The rib shortening of two-thirds of a foot per thousand feet due to all causes is thought to be fairly high and might, under certain conditions, be reduced.

7. In the solution of the problems shown in the various figures of this paper the hinge has been assumed at the top of the footings. Actually there would probably be very little difference whether it had been assumed at the top or bottom of the footings providing the designer is consistent.

8. Nothing has been said in this paper regarding the foundation conditions since the object of the paper was to present a mathematical analysis. It will be noted that with short stubby end members thrusts of real magnitude may be developed. Under these conditions it will be necessary to build into the foundation a resistance to this thrust to prevent undue movement at the piers. In the calculations as here presented a small amount of movement has been provided in the term Δ. Probably it would be on the safe side to make some such provision unless rock foundations were actually encountered making it unnecessary.

9. No attempt has been made to combine the moments as computed to give design conditions nor have shear diagrams been drawn. The most important thing is the determination of the indeterminate moments and when these have been found the shear and moment combinations are a relatively simple matter.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by July 1, 1936.*

Publication of Discussion Deferred

Publication of discussion intended for this issue is now scheduled for the May-June issue, under the following titles:

Inspection of Concrete

Placing Concrete by Means of Vibration

Properties of Mortars and Concretes Containing Portland-Puzzolan Cements

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Ordering, storing, bending, and fixing steel reinforcement

A. W. Legat, *Concrete and Constructional Engineering*, Vol. 31, No. 2, Feb., 1936, p. 122-137.
Reviewed by Glenn Murphy.

Discussion of methods of handling reinforcement. Illustrations and details of methods are given.

Types of concrete floors

Concrete and Constructional Engineering, Vol. 31, No. 1, Jan., 1936, p. 79 et seq., 15 pages.
Reviewed by Glenn Murphy.

Descriptions and drawings of several special systems of lightweight concrete floor construction.

Temperature stresses in chimneys and tanks

H. Carpenter, *Concrete and Constructional Engineering*, Vol. 31, No. 2, Feb., 1936, p. 105-113.
Reviewed by Glenn Murphy.

This article presents a method for calculating the stresses due to changes in temperature on one side of a restrained reinforced concrete member. Several examples are included.

Action of sea water on reinforced concrete

Concrete and Constructional Engineering, Vol. 31, No. 1, Jan, 1936, p. 73.
Reviewed by Glenn Murphy.

Summary of the results as given in the Report of the Building Research Board for 1934 concerning the inspection of several series of experimental reinforced concrete piles exposed in sea water. Different grades of cement and various mixes were used in the series.

Location of joints in concrete road slabs

E. Buhlmann, *Die Betonstrasse*, Vol. 11, No. 2, Feb., 1936, p. 38. Reviewed by Inge Lyse.

This article presents a theoretical analysis of the function of the joints and introduces an effectiveness ratio. It is found that the criterion for the effectiveness of a joint depends upon the so-called effectiveness ratio and the permanent set and the lateral deformation of the filler used in the joint. The compressive stresses in the concrete resulting from faulty joint construction have also been studied.

New methods of wall construction

Concrete and Constructional Engineering, Vol. 31, No. 2, Feb., 1936, p. 141-144.
Reviewed by Glenn Murphy.

Discussion of construction of a seven story block of flats built of reinforced concrete, but having the external appearance of a brick building due to the use of a special type of precast concrete wall panel. The walls consist of 5 in. of reinforced

concrete faced on the outside with precast panels 3½ ft. by 3 ft. by 2 in. The panels are made of 1-in. colored concrete bricks with a 1-in. reinforced mortar backing.

Influence on modulus of elasticity of concrete

A. HUMMEL, *Die Betonstrasse*, Vol. 11, No. 3, March, 1936, p. 61. Reviewed by INGE LYSE.

The problem of decreasing the occurrence of cracks in concrete and reinforced concrete construction has recently been given added importance particularly in concrete highway construction. In this article a thorough review is presented of most of the outstanding research work on modulus of elasticity of concrete. The recent publications by Glanville, Abeles, Yoshida and others have been carefully studied and the most important results reprinted.

A lightweight aggregate produced from slate waste

E. H. COLEMAN, *Concrete and Constructional Engineering*, Vol. 31, No. 1, Jan., 1936, p. 47-51. Reviewed by GLENN MURPHY.

A lightweight material produced by heating slate is described. Certain types of slate will expand to seven times the original thickness perpendicular to the cleavage planes, producing an aggregate with a density as low as 32 lb. per cu. ft. Tests made at the Building Research Station "indicate that the expanded slate compares very well with the best of the existing materials used as aggregate for lightweight concrete."

Newest concrete equipment for the German highway system

T. v. ROTHE, *Die Betonstrasse*, Vol. 11, No. 3, March, 1936, p. 49. Reviewed by INGE LYSE.

A discussion of the various types of equipment used in the construction of the German concrete highway system. Planers, rollers, and various tamping and vibration methods for the preparation of the foundation are discussed. Of special interest are the so-called automatic tampers. Pavers and finishing equipment are also described, and a considerable space is devoted to discussion of various types of concrete vibrators and to machines for construction of slab joints.

French build high dam in narrow limestone canyon

ROBERT A. SUTHERLAND, *Engineering News-Record*, Vol. 115, No. 21, Nov. 21, 1935, p. 706-9. Reviewed by N. M. NEWMARK.

Sautet Dam on the Drac River, in the French Alps, serves as a regulating storage for a number of power plants. The height is 414 ft., and the crest length only 263 ft. The dam consists of a comparatively slender constant angle arch backed on the downstream side by a solid mass of lean concrete to support the canyon walls and to protect the dam proper from possible falls of overhanging rock. The total capacity of the reservoir is 106,000 acre-ft.

A combined water tank and chimney

Concrete and Constructional Engineering, Vol. 31, No. 1, Jan., 1936, p. 15-17. Reviewed by GLENN MURPHY.

This article presents the design and construction features of a reinforced concrete chimney which carries a 5000-gal. water tank at the top. The chimney is 54 ft. high, with an internal diameter of 2 ft. 7 in., and an external diameter of about 5 ft. It is reinforced with two rings of vertical rods, and internal and external loops at 9 in. centers vertically. The tank has an external diameter of 13 ft. 6 in., and is 9 ft. 2 in. deep inside. The bottom of the tank consists of a slab 18 in. thick, which is reinforced by cantilever rods off the shaft, and by circumferential steel.

Road-base stabilization with portland cement

W. H. Mills, Jr., *Engineering News-Record*, Vol. 115, No. 22, Nov. 28, 1935, p. 751-3.
Reviewed by N. M. Newmark.

Field experiments are described on test sections of cement-stabilized-soil bases for bituminous surfacing. Seven test lengths of road in different localities were constructed during 1934 and opened to traffic. All of the field experiments are in good condition, but definite conclusions cannot yet be formulated. The cost per square yard of the test sections ran about 27 cents for cement, 10 to 20 cents for placing the cement, and 5 to 30 cents for the bituminous wearing surface.

Construction of dowels at highway joints

E. Goerner and H. Leussink, *Die Bautechnik*, Vol. 14, No. 9, Feb. 21, 1936, p. 134-137.
Reviewed by Inge Lyse.

A well illustrated article describing the various types of dowel construction used in highway joints with discussion of their merits and shortcomings. Ordinary simple dowels as well as dowels with various kinds of cross-ties and supporting devices are included, together with different types of provisions for free movements of the concrete slabs. In discussing the length of the dowels the authors recommend 25 to 30-in. dowels. Difficulties in proper placing and holding of the dowels are pointed out and attention is called to the fact that an improperly placed dowel is not only worthless but even detrimental to the joint.

Successive elimination of unknowns in the slope deflection methods

John B. Wilbur, Asst. Prof. C. E., M. I. T., *Proc. Am. Soc. C. E.*, Vol. 61, No. 10, Dec., 1935, p. 1463.
Reviewed by H. J. Gilkey.

The formal solution of a large number of simultaneous equations may be avoided by successively expressing all the unknown elements in terms of a few unknowns. The method is illustrated for continuous beams, building frames with shallow wind braces and the Vierendeel truss. The method does not actually avoid the solution of simultaneous equations but rather eliminates the unknowns progressively. The methods used may be extended to other structures solvable by the slope deflection method.

Standard specification for cast stone

Concrete Building and Concrete Products, Vol. 9, No. 1, Jan., 1936, p. 7. Reviewed by J. C. Pearson

The Cast Concrete Products Association (England) has recently issued a specification for cast stone. The usual limitations for cement, aggregates and pigments are included, with certain requirements of finish for both faced stone and stone of uniform composition. Four days' damp curing is to be followed by an additional 14 days under cover. Six-inch cubes at 21 days must have a minimum strength of 2000 p. s. i., which seems low in comparison with American and Australian requirements of 5000 p. s. i. at 28 days. An appendix gives recommendations for setting mortar, which is a lime mortar with or without a gaging of cement.

Thousands of holes grouted under Norris Dam

Engineering News-Record, Vol. 115, No. 21, Nov. 21, 1935, p. 699-701. Reviewed by N. M. Newmark.

The elaborate plan of grouting operations carried out in the foundations and abutments of Norris Dam is described. Holes up to 5½ in. in diameter, spaced 10 to 20 ft. apart, were drilled to depths of 30 to 50 ft. The holes were washed thoroughly, then filled with grout under a maximum pressure of 30 p. s. i. Rock condi-

tions in the larger drill holes were examined by means of a periscope lowered into the hole. In addition, 36-in. core drill holes were put down at various places enabling large-scale determinations of the results obtained by grouting. It was concluded that the foundation was completely sealed.

New Isar bridge in Bad Tolz

L. PISTOR, *Die Bautechnik*, Vol. 14, No. 9, Feb. 21, 1936, p. 129-134. Reviewed by INGE LYSE.

The design of this bridge won the award for the author, in a competition to which eight other engineers were invited to submit designs. The maximum stresses permitted were 17,000 p. s. i. for reinforcement and $\frac{1}{3.5}\ f'_c$ for concrete, where f'_c represents the 28 days strength. The bridge consists of four spans of about 78 feet each. Each span has six reinforced concrete girders, 6⅔ feet on centers with two transverse beams. Provisions were made for free longitudinal movement on each pier, thus eliminating any continuity. Detail description is given of the construction of the bridge, which when completed, will cost 450,000 reich marks.

Compounds in portland cement

R. H. BOGUE, National Bureau of Standards, Washington, D. C. *Industrial and Engineering Chemistry*. Vol. 27, No. 11, Sept., 1935, p. 1312-1316. Reviewed by ROY N. YOUNG.

This paper was one of a number included in a symposium on Materials of Construction in the Building Industry. In a readily understandable fashion it deals with the development of cement chemistry and briefly describes methods employed in the identification of most of the compounds in portland cement. The establishment of the parts which the various compounds play in affecting the properties of portland cement are discussed. An application of the results of the researches mentioned has, in the last decade, brought about important improvements in the efficiency of cement and durability of concrete.

Trinidad central water supply scheme

Concrete and Constructional Engineering, Vol. 31, No. 2, Feb., 1936, p. 115-121. Reviewed by GLENN MURPHY.

Description of the design of two covered concrete reservoirs each of 3,000,000 gal. capacity, being built as a part of the water supply system for Trinidad Island. The reservoirs are 200 ft. by 160 ft., each divided into two independent parts by a center partition. The roof is supported by steel columns encased in concrete. In constructing the floors of the reservoirs strips of bituminous sheeting 12 in. wide were laid on top of the three-inch thickness of lean concrete (which made up the lower layer of the floor) at intervals of 18 ft. in one direction and 20 ft. in the other, and on this the 6-in. floor slab was cast. A vertical joint filled with bitumen was left around each 18 by 20-ft. floor panel.

Aggregates for Grand Coulee dam

EDMUND SHAW, *Rock Products*, March, 1936, p. 30-41. Reviewed by ROY N. YOUNG.

Mr. Shaw, Contributing Editor of Rock Products, presents a vivid and detailed description of the treatment and handling of aggregates for Grand Coulee Dam, starting from the deposit and following through the various operations to the storage. The article is augmented by numerous photographs and drawings. (See also "Aggregate Production for Grand Coulee Dam" by Gordon F. Dodge—A. C. I. JOURNAL, January-February, 1936.)

Following this article (p. 41) is a review by Mr. Shaw of a paper given by Anthony Anable, Engineer of the Dorr Company, at a recent meeting of the American Institute of Mining Engineers "The Preparation of High Specification Concrete Sand at Grand Coulee Dam." It deals principally with sand classification.

Determination of grain shapes of aggregates

KURT WALZ, *Die Betonstrasse*, Vol. 11, No. 2, Feb., 1936, p. 27. Reviewed by INGE LYSE.

The question of shape of aggregate particles becomes an important one when sieve analyses are used as the basis for designing concrete mixes. With the same amount of material between different sieve sizes the actual sizes of particles in the aggregates may vary considerably when changing from well rounded gravel to elongated and flat pieces of crushed stone. This variation in shape of particles has been found to produce large variation in the water requirement for concrete mixes and the investigation reported in this article was made with the view of classifying the various shapes. The classification was based onthelengthofthe three axes of each particle and the comparison made on the ratios between the two larger and the smaller diameter. Thirty-nine groups of aggregates were carefully measured and studied. The results indicated that visual inspection may often be misleading for judging the average shape of the aggregate particles.

Tests of mortars for reinforced brick masonry

M. O. WITHEY AND K. F. WENDT, *Proceedings*, A. S. T. M., Vol. 35, 1935, Part II, p. 426 AUTHOR'S SYNOPSIS.

Tensile and compressive strengths, linear change measurements, absorption, freezing and thawing, and autoclave tests, also workability ratings, are reported. Most of the tests were made on 1:3 and 1:4 portland-cement mortars rendered more plastic by additions of lime or pulverized clay. The influence of five curing conditions, five lime hydrates, three varieties of clay, and differences in proportion of admixture are included. Approximately 82 mixes and 4000 test specimens are represented in the reported data. The large linear changes of the dolomitic lime-cement mortars under continuously wet curing and of the clay-cement mortars in dry air are shown. The data indicate the suitability of 1:4 mixes with proper addition of lime or clay for moderate exposure, but for high strength or severe exposure 1:3 or 1:2½ mortars with clay or lime additions appear preferable.

The chemistry and physics of concrete shrinkage

R. W. CARLSON, *Proceedings*, A. S. T. M., Vol. 35, 1935, Part II, p. 370. AUTHOR'S SYNOPSIS.

A simplified explanation is given of the manner in which the gel resulting from the hydration of portland cement causes shrinkage upon drying. Attention is directed to the fact that all concretes shrink more than would be expected from the individual behaviors of cement paste and aggregate, and from the relative dimensions of each in the concrete. Although many possible explanations suggest themselves, it is indicated that voids around the aggregate particles and within the aggregate may be the most important factors causing concrete to shrink more than would be expected. Data are presented showing the effect of size of mass and distance from an exposed surface upon the shrinkage of concrete. It is shown that in large masses the tendency to shrink is limited to the region within a few inches of the surface, even after 4 months of drying. Even a member of no more than one foot thickness may shrink but a fraction as much as a small bar during an average drying season.

Shrinkage of concrete

INGE LYSE, *Proceedings*, A. S. T. M., Vol. 35, 1935, Part II, p. 383. AUTHOR'S SYNOPSIS.

This paper presents the results of an investigation of the use of ordinary 3 by 6-in. concrete cylinders for volume change observations and of observing these changes by

a simple apparatus consisting of an Ames dial attached to a metal base, and of a study of some of the factors which contribute to volume change in concrete. The factors investigated were quality and quantity of the cement paste used, length of moist curing, and different cements and aggregates. The 3 by 6-in. cylinders were found to serve well for shrinkage observations and the measuring apparatus used gave satisfactory results. The quality of the paste showed little effect upon the shrinkage, while the quantity of the paste contributed approximately in proportion to its percentage of the volume of the concrete. The length of moist curing had practically no effect upon the shrinkage. The high-early strength portland cement produced more shrinkage than did the standard portland cement, particularly in lean mixes, but in terms of shrinkage per unit of strength, the high-early-strength cement proved superior. Fine aggregate showed more effect upon shrinkage than did coarse aggregate.

Temperature control for a small moist curing room

The National Sand and Gravel Bulletin, Dec., 16, 1935 COURTESY HIGHWAY RESEARCH ABSTRACTS.

The moist curing room at the National Sand and Gravel Association Laboratory is about 8 by 8 by 8 ft. A temperature of 70° ± 1 degree and a humidity of 100 per cent are desired. A simple device has been designed and built by C. E. Proudley, Assistant director, which maintains the temperature more uniformly than guaranteed by commercial companies bidding for the installation.

The apparatus includes an ice chest, electric heating units controlled by a thermostat, a fan, and (at present) a mine spray. Ice in the chest cools the room to 70°F. and slightly below. Immediately when the temperature falls below 70°, the thermostat cuts in the electrical heating elements which stay on until the temperature is raised to about 70.5°. The fan circulates the air to maintain uniform temperature throughout the room.

The operating cost includes a small charge for electricity and, during hot weather, about 50 to 80 cents per day for ice. Cost of materials for construction was about $20. Labor was done at odd times by the regular staff.

Tests on ten-year-old concrete

M. O. WITHEY, *The Wisconsin Engineer*, Feb., 1936. AUTHOR'S ABSTRACT.

Reports results of tension tests on mortars and compression tests on concrete made with four different brands of portland cement. In the concrete tests three types of aggregate were used, two consistencies, and in some of the tests the effects of three water ratios were studied. Part of the specimens were subjected to curing out of doors and part were cured in laboratory air. The neat cement and 1:1 standard sand mortar specimens exhibited marked variability in tensile strength due to out door curing over the ten year period. The concretes showed a good increase in strength when cured unprotected on relatively dry ground in the climate of Madison. Concrete cured indoors at low humidities showed very low rate of increase in strength after 3 months. The actual difference between strength of concrete of dry consistency and that of wet consistency increased with time. At the five and ten year periods the mortars in concretes made of cements highest in dicalcium silicate had slightly higher strengths than those made of cements highest in tricalcium silicate. No material difference in concrete strengths appeared to result from the type of aggregate used.

Steam curing of concrete blocks

A. W. WOLJENSKI, *Beton Stein Zeitung*, Vol. 2, No. 3, Feb. 10, 1936, p. 33-35.

Reviewed by INGE LYSE.

A summary of an investigation carried out at Moscow, U. S. S. R., during 1934. The experiments were made on blocks about 5 by 5 by 8 in. machine-made from

dry concrete mixture. The aggregate consisted of crushed limestone and natural sand. Since the sand contained from 3 to 7 per cent clay a supplementary study was made of the effect of the clay on the strength of steam-cured concrete specimens. The results showed that the strength increased with the increase in clay content in the sand up to the maximum of 20 per cent used. The steam curing was applied for 12 hours (2 hours for increasing the steam pressure, 8 hours under constant pressure, and 2 hours for gradual decrease of pressure). The length of increasing and decreasing the steam pressure was found to have an important effect upon the cracking of the blocks. The 2-hour periods were found to give the best results. The tendency towards cracking was found to be dependent upon the cement content of the blocks, the higher the cement content, the greater was the tendency to crack. The effectiveness of the steam curing was more pronounced for lean than for rich mixes. While normal curing gave strengths from 1400 to 1700 p. s. i. at 28 days, the steam curing produced strengths of 2100 to 3200 p. s. i. Attention is called to the difficulty of securing laboratory conditions comparable with actual production methods.

Determination of modulus of elasticity and Poisson's ratio of concrete at ages of fourteen days to four years

L. H. KOENITZER, *Proceedings*, A. S. T. M., Vol. 35, 1935, Part II, p. 399. AUTHOR'S SYNOPSIS.

This paper is the continuation of the work of P. M. Noble reported at the annual meeting of the Society in 1931. (Vol. 31, Part I, p. 399) Tests were made on concrete mixes containing sand-gravel aggregate, soft limestone, sandstone, flint gravel, crushed flint aggregate, and a dense limestone. The water-cement ratios varied from 0.6 to 0.9 with slumps from 0 to 8.5 in. The modulus of elasticity and Poisson's ratio were determined on 165 6 by 12-in. cylinders in a moist condition at ages of 14, 28, 56, 112, 380 and 1480 days. Tests were also made on the same specimens at the age of 360 and 1460 days in a dry condition. Tests show that the modulus of elasticity of plain concrete at the end of 4 yr. is about the same as at the age of 14 days. Poisson's ratio for the mixes studied indicate a rapid increase for the first 56 days and then a gradual decrease until at the age of 4 yr. the ratio was in most cases equal to or less than Poisson's ratio at 14 days. Sand-gravel aggregate concrete failed to increase in strength, the strength at the age of 4 yr. was in many cases equal to or less than at the age of 28 days. Sand-gravel aggregate concrete specimens when tested wet gave a lower modulus of elasticity than when the specimens were tested dry. The opposite was true for coarse aggregate specimens. Aggregates have an important influence on the elastic properties of the resulting concrete.

Dynamic effect on road slabs

A. RAMSPECK, *Die Betonstrasse*, Vol. 11, No. 2, Feb., 1936, p. 32. Reviewed by INGE LYSE.

Heavy traffic often produces dynamic effects which may be critical for the strength of the road slab. In the study presented in this article special attention is given to the vibration set up in a slab due to the impact produced by the dropping of weight on the slab. The vibration curves thus produced in the slab may be assumed to approximate the following equation:

$$y = A.sin\ 2\pi \frac{x}{\lambda}$$

where λ is wave length, A amplitude of the wave, and y is the deflection of the slab at a distance x. The stress produced by this deflection is:

$$6 = E \cdot \frac{h}{2} \cdot \frac{4\pi^2}{\lambda^2} \cdot A.sin\ 2\pi \frac{x}{\lambda}$$

where h is the thickness of the slab and E is the modulus of elasticity of the material in the slab. The maximum stress will be:

$$\sigma_{max} = E \cdot \frac{h}{2} \cdot \frac{4\pi^2}{\lambda^2} \cdot A$$

The major problem was to study the conditions for the wave length and the amplitude. An extensive investigation was carried out on concrete slabs; other slabs will be studied later. Slabs of different thicknesses and supported on different types of foundations were included in the observations. The results are presented in diagrams and showed that the thickness of the slab, as well as the character of the foundation, had an important effect on the vibration.

Research on concrete disintegration

H. S. MATTIMORE AND G. A. RAHN, *Proceedings*, A. S. T. M., Vol. 35, 1935, Part II, p. 410, 420.
AUTHOR'S SYNOPSIS AND CONCLUSIONS.

This paper presents the results of a research project undertaken to determine the cause of an abnormal rate of disintegration of concrete structures in northern Pennsylvania. 45 experimental concrete headwalls were constructed in the same location as the original walls which had failed, and these were closely observed and their resistance to exposure recorded. Two different sands, and two different cements with the same coarse aggregate were used; also the cement and aggregate proportions and water-cement ratio were varied. The inspection record, with the available detail data on material qualities, concrete proportioning, compressive strength, pH determinations on surrounding soils and daily recording thermometer records over the entire exposure period furnishes information rarely, if ever, available on studies of field concrete. The periodic inspections made it possible to record the development of disintegration and to establish a rating on stages of disintegration, which it is hoped will be of value in recording data on similar investigations. Some of the probable causes for disintegration are discussed in the paper, but what is believed to be of more value is that sufficient data are furnished so these causes can be checked or other conclusions drawn. Conclusions:

Extremes in temperature between exposed section and buried part of wall subject these specimens to high stresses. Saturated ground conditions with rather frequent changes in temperature produced numerous cycles of freezing and thawing during the 4½ yr. of exposure. Cement A and sand C are definitely indicated as factors affecting adversely the durability of concrete under this condition of exposure. Table V giving the pH value of soils in contact with these walls is convincing that deleterious acid or alkali action is not a factor in this disintegration.

Heat of hydration of partially prehydrated cements

Technical News Bulletin, National Bureau of Standards, Nov., 1935, p. 114.
COURTESY HIGHWAY RESEARCH ABSTRACTS.

As a means of decreasing the amount of heat developed in mass concrete by the hydration of the cement, a study has been undertaken at the Bureau of Standards to determine the effect of prehydration of the cement. Two different types of cement were treated with steam at atmospheric pressure so that approximately 3 and 5 per cent of water, respectively, as determined by loss on ignition of the resulting product, was taken up by the cements.

The heat of hydration was determined by the heat of solution method. Test ages were 7, 28 and 90 days, and 1 year. Three curing conditions were used: (1) 70°F. for length of test, (2) simulated adiabatic curing (70°F. for first 24 hours, then 150°F

to end of test), (3) a combination of these two types wherein, at 7, 28 and 90 days, specimens were removed from 150°F. storage and put in 70°F. storage for the remainder of a year.

Prehydration with 3 per cent water reduced the average heat of hydration of the standard portland cements cured at 70°F. by 14, 12 and 6 per cent for the respective ages of 7 and 28 days and 1 year. Corresponding reductions in heat for 5 per cent of water were 28, 27 and 16 per cent. Under simulated adiabatic storage, the same order of decrease in heat of hydration was found as for the 70°F. storage.

The respective prehydrations of high-early strength cements caused about two-thirds the decrease in average heat of hydration as was found for the standard portland cements.

It is stated, that, considering only the heat of hydration values, a low heat cement can be prepared by prehydrating with approximately 5 per cent of water. The prehydration also very materially improved the resistance of the cement to sodium sulphate solution as measured by the Merriman test.

Seasoning of portland cement at elevated temperature

Paul S. Roller, Nonmetallic Minerals Experiment Station, U. S. Bureau of Mines, New Brunswick, N. J. *Industrial and Engineering Chemistry*, March, 1936, p. 362-369. Reviewed by Roy N. Young,

The apparatus for seasoning ground clinker, procedures and results of tests are described in detail. In the seasoning process ground clinker is subjected to atmospheres of steam or mixtures of steam and air at temperatures above the dew point under closely controlled conditions. The rate of absorption of water under these treatments decreases rapidly with time and increases with decrease of temperature. At temperatures only slightly above the dew point, rate of absorption becomes very rapid. Increased fineness increased water absorption over one and one-half times faster than the increase in specific surface. Studies were made of normal consistency, time of set and strengths as affected by different degrees of seasoning. As the amount of absorbed water increased up to an optimum point, the normal consistency decreased, while the time of set and strengths increased. Beyond this optimum point (overseasoning) the effects were reversed, particularly with respect to normal consistency and strength, resulting in higher normal consistency, lower strengths and in some cases longer setting times. In general the absorption of CO_2 resulted in higher normal consistency and shorter time of set—flash set in some cases. The seasoned cements offered greater resistance to changes which normally result from the absorption of CO^2. The author holds that in the commercial production of cement some degree of seasoning occurs or else retardation of set by gypsum would be unsuccessful. The necessary water for this seasoning may be derived from the gypsum which is ground with the clinker. Overseasoning may occur due to additional absorption of water from other sources. Controlled seasoning therefore presents the possibilities of a more uniform product with higher strength, a more stable cement with respect to influence of moisture and CO^2 absorbed during storage and the elimination of gypsum as retarder, together with any of its accompanying evils.

Influence of catalysts in the production of high-strength cements

W. Watson and Q. L. Craddock, *Cement and Cement Manufacture*, Vol. 9, No. 1, Jan., 1936, p. 13-19. Reviewed by J. C. Pearson.

In this review of the effects of admixtures on raw materials and finished cements, the authors add another to their numerous contributions of similar nature, published within the last year or two in this Journal. All investigations seem to show that good clinkering is of prime importance in the production of good quality cements.

This result is obtained primarily by intensive burning, sometimes aided by the use of suitable fluxes to lower the clinkering temperature. Rapid cooling of clinker is desirable, the effect being analagous to the quenching of blast furnace slag in improving its hydraulic properties.

Among catalysts added to the raw mix, which include alumina, iron oxide, fluorspar, borax and alkali salts of various types, iron oxide and fluorspar have been attended with most success. Catalysts have also been added by grinding them with the clinker. Gypsum and calcium chloride not only serve to control the setting time but they also add strength. Calcium chloride has been used principally as an admixture to the finished cement, and the authors present a well balanced selection of references setting forth the pros and cons for this material. The effects of numerous other metallic chlorides are cited, indicating that chlorides of the alkali metals and some others tend to increase strength, while those of the heavy metals are injurious.

Numerous sodium salts have been added to quicken set or improve the early strength of concrete. References are given to the effects of carbonate, bisulphite, bisulphate, phosphate, borate and silicate. The effects of hydrated lime, slag, trass, spent shale, diatomaceous earth and powdered iron are mentioned. The authors express the opinion that further investigation of the influence of catalysts on the process of clinkering promises far better results than additions to a cement obtained from comparatively poorly clinkered material.

Volunteers Wanted for Foreign Language Reviews

Institute reviewers (without compensation) regularly scan foreign language technical publications for contributions presenting something new and significant in the field of cement and concrete. Of the outstanding papers and reports they write reviews trying to preserve the essence of the original in very small space with some guiding comment on the importance of each contribution. Sometimes there is a vacancy in the corps of reviewers. Reviewers understand that their work must be reasonably up to date. When they can't continue the work regularly we look for someone else to take on the task. We should be glad to list volunteers for such work as the occasion arises. A reviewer must be conversant with the literature of the field so that he may recognize worth and novelty. "Old stuff" is to be avoided. An outstanding contribution is almost sure to go the rounds of many publications in various stages of "rehash." Many publications contain little but "rewrite" stuff. A. C. I. reviewers aim to catch something worth while close to its source. If you are in a position to read foreign technical publications coming to the Institute; if you enjoy such work and want this opportunity to keep your languages from getting rusty; if you want to keep up with the literature of cement and concrete and welcome an opportunity that may assist in both the language and the subject matter; if you have a reasonable facility in separating the significant from the superfluous and boiling down many words into few—we should be glad to have you volunteer for this work as it becomes available. State your language qualifications as among publications in French, German, Italian, Russian, Scandinavian and Spanish.

Concrete Restoration in Water Impounding Structures*

BY J. LAMPRECHT†

INTRODUCTION

The restoration of concrete will be of interest to anyone charged with the maintenance of buildings or other structures of concrete and the problems encountered are so difficult of solution that those identified with original concrete construction may also profit by their recital, for probably 95 of every 100 instances of concrete disintegration can be traced definitely to bad aggregate, or bad mixing, or bad placing.

If the concrete we place today is made properly, of good materials, and proper consideration is given to local conditions, it will not need restoration in 10 years—or 20 years—or even 30 years. It may require some maintenance, yes, but no restoration as we know it today, of structures 5, 10, 15 years old.

While this discussion will in general be confined to the subject of the restoration of concrete, particularly in water impounding structures, the hope may not be vain that it will draw attention to the superiority of prevention over cure in concrete generally.

For instance, look at Fig. 1. Familiar isn't it? Disintegration of concrete at the water line. We may note with some satisfaction that the concrete above and below this always vulnerable area is in much better condition, but we can't be complacent even about that, for if the concrete had been made properly in the first place, it would have stood up even at the water line.

A dam spillway in the Adirondack foothills, where winter brings plenty of ice, snow, and temperatures lower than 35° below zero, is shown in Fig. 2. Yet the plane of the front face of this spillway is probably within a small fraction of an inch of the plane of this face when it was built in 1900. Fig. 3, from a photograph of the structure taken during construction, shows that the material was placed in thin lifts and so dry that forms were needed only as guides. Good aggregate

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936.
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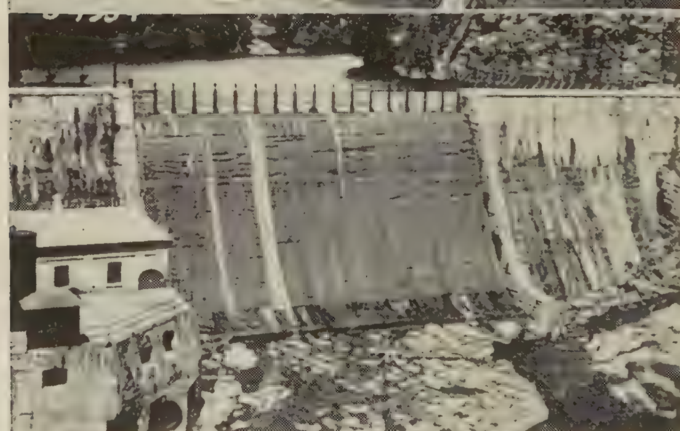

FIG. 1—DISINTEGRATION OF CONCRETE AT WATER LINE
FIG. 2—SPILLWAY IN GOOD CONDITION AFTER 35 YEARS

was used, the concrete was carefully mixed, there was no water excess, and it was well placed.

Fig. 4 is of another dam spillway not more than 60 miles from the one seen in Fig. 2 and 3. It was built in 1921, not 1900, and look at it! *Modern* concrete—with probably all the known ills that result from bad aggregate, bad mixing, and bad placing—typical of thousands of structures in this country, and probably all over the world, under similar climatic conditions, while structures like that shown in Fig. 2 are exceptional. Why? Simply because construction speed, cheap

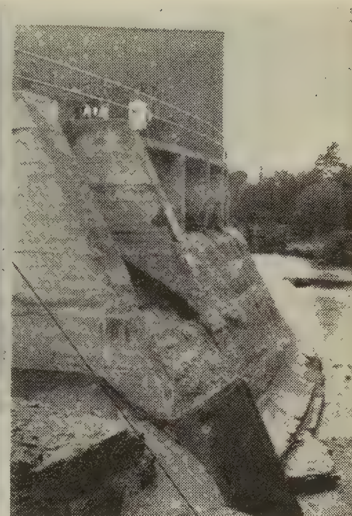

FIG. 3—THE WORK SHOWN IN FIG. 2 AS PLACED, VERY DRY
FIG. 4—(RIGHT) DISINTEGRATION SINCE 1921

material, cheap labor, play too important a part in a sham economy, that begets only grief to the owner for the life of the structure. An economy so often sham even in the first instance, for it costs little more, and sometimes less, to make good concrete, than to make the sorry material that sometimes masquerades under that name.

Enough poor concrete has been placed to provide us a vast heritage of "sick" structures. They will continue with us for many years to come. What is to be done about this current bad concrete? Can anything be done of a permanent nature, or even of a semi-permanent nature? Or must we allow the existing admittedly bad concrete to continue disintegration to inevitable disuse or replacement of the structure.

WHY RESTORE?

Now there are two basic reasons which provide the urge for the restoration of concrete. One has to do with its usefulness, the other with its appearance. The structure's usefulness is, of course, the more important, although not always the actual or sole cause of remedial action. Frequently attention is drawn to the condition of a structure by its appearance, which sometimes over-accentuates, sometimes minimizes its dangerous features.

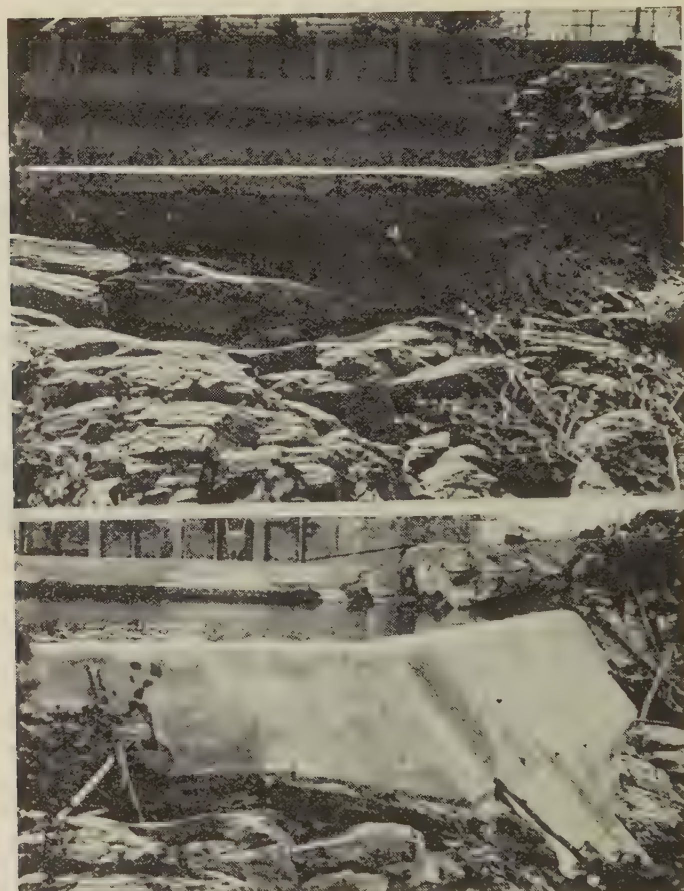

FIG. 5 AND 6—BEFORE AND AFTER RESTORATION OF DOWN STREAM FACE

As an example of appearance over-emphasizing a bad structural condition, I cite a concrete dam subjected at flood to a total head of 18 to 20 feet at its base, which before restoration had all the appearances of a gravel bank, on its downstream face—and not too good gravel at that. One look at this downstream face (Fig. 5), and you ask "What holds it up?" Yet, although a maximum depth of three feet of concrete had spalled or eroded from the face, there was still a substantial amount of dam left, and on close inspection the remaining material proved to be in very fair condition, so that the dam was still a stable and useful structure. Fig. 6 shows the same structure after restoration in 1934.

Fig. 1, shows a structure that definitely *does* require attention for its stability. It was built in 1921 and if 14 years can do that much, one guess is as good as another as to the result of 10 more years of the same set of conditions.

The downstream face of another dam (Fig. 7) looks bad, but disintegration actually is only "skin deep." Good sound concrete is within an inch of the surface, and unless appearance means much, which it usually doesn't on structures back in the woods as this one is, nothing need be done on *that* face. Further disintegration may confidently be expected to be slow—certainly where entrance of moisture from the upstream side has been eliminated or reduced to a minimum by completely waterproofing that face, as has been done in this case.

Generally, appearance is a factor in undertaking restoration work only where structures are in general view, or where appearance obviously is important, as in buildings, bridges, etc. When restoration is undertaken from a utilitarian standpoint, appearance of the finished work need not play too important a part, as will be brought out later. Certainly appearance is of secondary importance to utility and permanence.

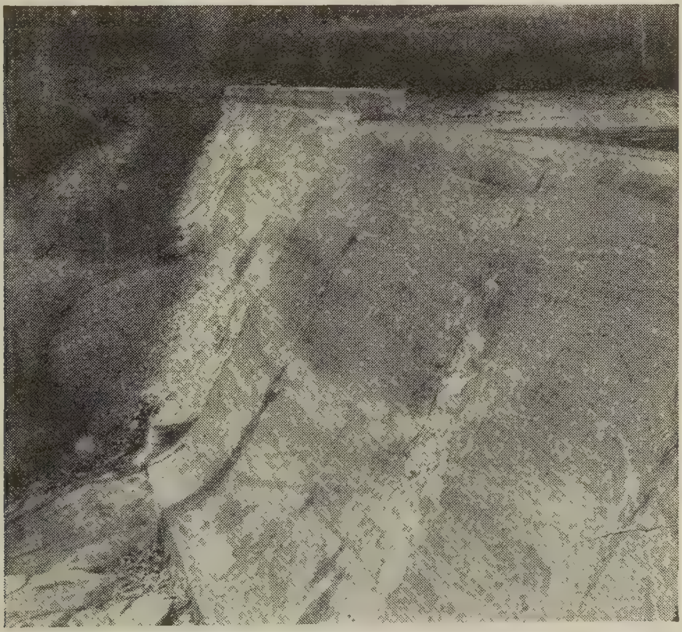

Fig. 7—A down stream face better than it looks

HOW RESTORE

Regardless of the reason for undertaking restoration work in any given instance, approval will be universal that the new work must be as nearly as possible as permanent as the parent structure. To be this, the restoration work must:

a. Support all structural loads to which the use of the structure subjects it.

b. Adhere tenaciously and with certainty at all points to the original material.

c. Withstand erosion due to ice and water.

d. Provide for the direct and indirect results of extremes of temperature, particularly as reflected in expansion and contraction of the restoration material and of the parent structure.

e. Be waterproof.

This may sound trite. *Of course*, you say, all these things must apply to new work replacing old. How else could the expense of restoration work be justified? It *is* more or less axiomatic. But have you ever actually attempted to do this kind of restoration job? Have you ever tried to apply to an existing concrete structure a superficial layer of new material—texture, ingredients, characteristics, etc., all differing from those of the existing structure, in a manner that will insure its structural stability, its adherence to the original structure, its weather-worthiness, its waterproofness? It isn't easy. It isn't accomplished even in moderate degree in half or perhaps a third of all restoration jobs. It isn't accomplished 100 per cent in any restoration job for three reasons. First, we don't know how to do it. Second, we don't know we don't know how to do it. Third, we haven't seemed to care whether or not we know how to do it.

CONFESSIONAL

When the subject of the restoration of concrete first came to my urgent attention, about three years ago, I certainly knew little about it, but I knew that. I still don't know much about it, but I know that better, and that's progress. As with other general engineering or construction problems, I assumed, since concrete has always had bad examples, and presumably has always had to be restored, that my task was chiefly that of referring to the experiences of others. I felt reasonably sure that these would present a picture of my own problems and point inevitably to a single or composite solution based on many previous trials, experiments, difficulties, failures—someone trying this method and discarding it; someone else trying another method and discarding that. Maybe in a hundred different places, under a hundred different conditions—at any rate so that I'd have no trouble charting a course with pre-known results through my own difficulties.

And what did I find? Some reference to the general problem of concrete restoration, of course, chiefly in trade media. Some examples of what had been done in instances not comparable to the problems I faced. Practically no reference to restoration of concrete in dams, or similar water-impounding structures. Nothing of the manner in which such restoration work would stand up under given weather or use conditions. Nothing concerning what I am now sure are routine characteristics of restored concrete, or how its bad features could be suppressed by proper installation and materials, or how their effect could be reduced by proper maintenance.

Now perhaps this lack of material was not real and appeared to exist in my case only because I didn't know where to look for it. Or because the urgency of the work precluded the possibility of taking enough time to look for it. Maybe so. Yet, I have since found many who have pioneered in this type of work—spent the productive period of their lives at it practically. And I am not able to say now that the information I have since obtained from others, apart from that arising out of my intervening experiences herein related, would have been sufficient for my purpose even had it been available in the first instance.

Where is the grist of the mills of past concrete restoration work? If it has not yet been collected, analyzed, and summarized, it probably never will be, and its value will of course be lost. But there is no reason why current experiences and lessons should not be made available for that purpose. That is the only logical excuse for this paper, dealing as it does with only a small phase of the subject and relating to only a very few actual examples. It will fail even in that purpose, however, unless supported without delay and continuously by further data made available from similar sources.

Concrete restoration work is identified closely with the so-called "gunite" or "pressure-concrete" method of application, which supports a large, important and growing industry, inevitably allied with the cement industry. It is difficult to gage the present extent or probable growth of concrete restoration work, but if the condition of concrete structures everywhere is any criterion, activity along this line in the next few years will be sufficient justification for those industries to take that type of work out of the field of conjecture, and into one of relative certainty, by analyzing and specifying its proper functions, its best materials, its correct methods of application, and by determining the results that can confidently be expected from a combination of these.

THE PROBLEM—CONDITIONS, CAUSES, RESULTS DESIRED

Requisites for the solution of any given corrective problem include the determination of (a) the conditions of the problem, (b) their causes, (c) the results desired, and (d) the methods of accomplishing these results.

The actual condition of any concrete is not too apparent from a superficial examination of the surface. A thin surface spalling may make for a worse appearance than a more deep-seated difficulty. In many instances of surface spalling, the veneer of disintegrated surface material will come off in large areas if coaxed a little, and underneath will be found a thin layer of powder-like material overlying perfectly sound concrete, at an average distance of 1 in. back from the original surface. Only a concern for the appearance of such a structure would dictate restoration of its face.

On one such structure the water side doesn't look so bad, but in some areas it was necessary in the restoration process to remove a depth of as much as 18 in. of bad concrete before reaching a material satisfactory for application of new pressure-concrete.

Usually it will be possible to determine the approximate depth of disintegration by hacking away a small section of the surface with any sharp tool, such as a brickmason's hammer. Such a tool is also invaluable during the actual restoration work in determining by the sound of its blow on any concrete surface whether or not sound material has been reached. An unmistakable ring will indicate sound material; a hollow sound, loose material, a dull sound, "dead" material. It will similarly reveal any areas of restoration work not bonded to the base material.

A more satisfactory way of determining the condition of concrete below the surface would be by obtaining core samples, and it is surprising that the advantages of such a method apparently have not resulted in development of the proper tool. However, I have devised such a tool with Carboloy teeth, for providing a 1-in. core, and intended for use with an electric or air drill. When operated in a rigid vertical bearing attached to a three-legged base, with compressed air brought to the tool tips for removing the dust, good representative cores are obtainable.

Most concrete disintegration problems in water impounding structures originate with the scouring and seepage of water at the upstream face, and it is there that most of the restoration expense can be justified.

Inspection of the upstream face must, of course, be made when the water is down, with consequent expense due to loss of water, power or both. However, the general condition of the concrete on the upstream face below the low water line is invariably much better than in the areas above that line, so that an examination down to the normal low water line will generally reveal the worst conditions.

We may determine with as much pains as we like the condition of the concrete in a structure but unless we also obtain a clear idea of the causes of that condition, we cannot decide on the proper steps to correct it. Primarily the original quality of any given concrete has most to do with its life. Yet I hazard the statement that there is comparatively little concrete so bad that it would not have stood up satisfactorily under very favorable conditions—mild, dry, uniform climate, uniformity of use, etc. And correspondingly very little concrete so good that it would stand up under very unfavorable conditions—extreme climate, acid contact (via water or air, or directly), lack of uniformity of use, etc. It is, then, the combination of the quality of the material and the conditions applying in any particular case that determines its history.

Downstream face spalling may be due to direct infiltration of some moisture into that face, with alternate freezing and thawing. Evidence is lacking to support the idea that disintegration so caused is continuous and progressive. Rather it is probably at a minimum immediately after construction is completed until the skin surface is broken down somewhat, then progresses more or less rapidly through the thickness of mortar brought to the surface during construction by spading, and continues at a progressively slower pace with the disintegrated material actually in a protective role. In such case there is little justification for doing anything, unless appearances require action, and then the removal of the spalled material may be sufficient, although the addition of a surface waterproofing will, of course, prevent the recurrence of the spalling condition.

However the problem is rarely so simple. Usually downstream face spalling results from a combination of infiltration of water into that face, as just outlined, plus seepage of the impounded water at the upstream face through vertical construction or expansion joints, horizontal pour joints, cracks, porous concrete, etc. Obviously it would be better to ignore spalling on the downstream face and permit unrestricted seepage there than to attempt to stop it at that face by waterproofing, unless the upstream face is also waterproofed.

We reach then the conclusion that disintegration of concrete, regardless of the quality of the concrete, is due mainly to infiltration of

water, dissolving some of the cementing materials from it, or where the contact water contains certain harmful acids, etc., actually breaking down either the cementing materials or the aggregate, and, in cold climates only, due to freezing and thawing of this same moisture content, resulting in a mechanical breakup of the concrete from the downstream surface inwardly. If this is so, preventing the water from entering the structure should cure all these difficulties. It will. And nothing else will.

BASIC METHODS OF RESTORATION

How then can this essential thing be done? It is patent that either the mass of material must be made waterproof, as might be done by pumping grout into it through drilled holes, in which the efficacy of the method in any particular case is impossible of appraisal, or a waterproof covering must be placed over the entire water contacting or upstream surface. This latter method is the one currently employed and is the basis of most of the work referred to here.

How much of the water contacting surface should be so treated? Most dams or other water retaining structures have sluice gates or other facilities for unwatering down to a certain point. To get at the areas above this point for restoration operations involves therefore only the loss of power (due to loss of head and water) in power supply dams, or loss of water in water supply dams. Obviously, however, water may enter the dam structure below the normal unwatering line as far down as the rock line, or through the joint between the concrete and rock, or yet through seams in the rock upstream from the dam. Now it is impossible economically to close every such possible access of water to the structure. It is usually difficult to justify the expenditure necessary to unwater a structure down to the rock line along the entire length of the structure, to say nothing of attempting to prevent seepage through adjoining rock surfaces. This brings us to the following wholly practical conclusions:

1. That the point to which restoration in such structures should aim is the rock line.

2. That the complete realization of that aim should be controlled by its cost, remembering that while the cost of unwatering below the normal discharge level and of removal of the always present muck and sludge at the bottom, is exceedingly high, the actual advantages of waterproofing the areas so uncovered are few and less important, since:

(a) Experience shows that the concrete in these areas is in the best condition.

(b) The probability is that the bottom muck itself actually tends to waterproof the surface it contacts.

(c) It is impossible to prevent *all* water seepage since rock seams far upstream from the structure *may* carry water to the concrete base, and

(d) In all cases it is necessary in any event to anticipate discharge of some seepage from the downstream face, this seepage arising not only from the above possible points of weakness but also from such leaks through the actual waterproofing as develop due to our old alibi—"the human element," by making provision for the easy drainage of this seepage away from the structure after it has developed, by installation of weeps, etc.

VARIOUS METHODS OF RESTORATION

There are a number of current methods of tackling the actual problem of concrete restoration, or waterproofing. Suppose we enumerate first and then discuss some of the more important of them, describing the preparation of the existing surface, the restoration materials used, their application, the various finished surfaces available, the difficulties encountered—some common to all jobs, some peculiar to certain jobs—and how these various types have met the requirements listed in the five fundamentals listed on page 538.

The list includes:

(a) Wood-floated, steel-trowelled, cement-sand "plaster" covering of uniform thickness.
(b) Gravity placed concrete, dowelled and reinforced.
(c) Brush-applied, cement-sand washes.
(d) Pressure-concrete, applied either as a relatively thin covering, or in bulk, reinforced.

The possibility of adding to the aggregate of any of these, during the mixing operation, various manufactured materials intended to reduce or eliminate cracking, spalling, etc., will be discussed later.

THE "PLASTER" METHOD

Fig. 8 shows the top of a section of canal wall before restoration, and Fig. 9 the same section of wall 14 months after restoration by the "wood-floated, steel-trowelled, cement-sand plaster" method. The wall supports at the back, an earth fill reaching nearly to its top. Restoration was from the ground line at the back, up over the top and down on the face or water side to a point 9 ft. below the top, the wall below being in relatively good condition.

On this job all unsound material was removed by hand tools down to the point where the remaining surface was satisfactory for the application of the plaster coat. Since the work was done by contract, under a ten-year guarantee against "crazing, cracking or spalling," decision as to what was a "satisfactory" surface for application of the restoration material was left to the contractor.

After thorough cleaning of this surface, a thin "scratch" coat of cement-sand plaster (1:2) was trowelled on, and the next day a further

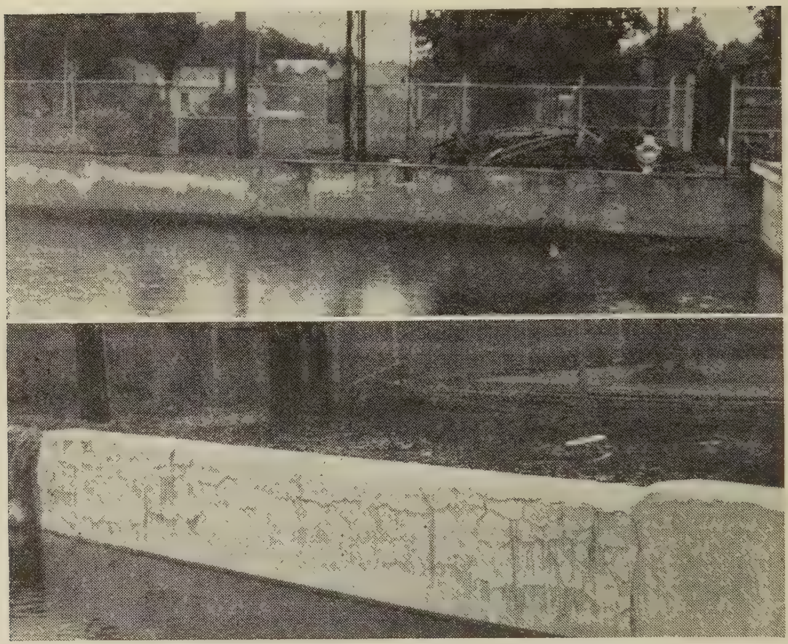

FIG. 8 AND 9—DISINTEGRATION AND CRAZING AFTER REPAIR

coat of the same 1:2 cement-sand plaster was applied, of an average thickness of ¾ in. It consisted of nothing but cement, sand and water, and contained no reinforcing. The material was placed as dry as possible (considering the normal tendency of the workman to ease its application and floating by addition of excess water) wood-floated, and finally energetically steel-trowelled to a smooth finish, hard to the touch of the finger pad, although capable when green of being nicked with any sharp object.

The final surface followed with smooth contours the approximate surface of the concrete wall after the unsound material had been removed. No weeps were used. The total cost was approximately 30 cents per sq. ft.

Just after the work was completed it was a beautiful job! But crazes and cracks soon began to appear above the water line, and this condition is becoming progressively worse. To date this cracking and crazing is not generally accompanied by loosening of the bond between the plaster and concrete, although some loosening has occurred, obviously due to incomplete removal of bad base material.

Only that part of this wall above the water line, as shown in Fig. 8

and 9 has been seen since restoration but it is believed that the portion below the water line lacks the cracking above that line. Note that the cracking is most prominent in the left hand section of this wall, which was the portion in the worst condition prior to restoration.

Fig. 10 shows the upstream face of a short section of the spillway structure on the opposite side from the canal wall shown in Fig. 8, just after restoration to rock by the same method. Fig. 4 shows the downstream face of this spillway in the same area before restoration and Fig. 11 just after. Fig. 12 is a close-up of the stop log groove in one of the platform piers before restoration, and shows the mechanical need for repairs.

The work of restoration on this spillway was carried out exactly as in the case of the opposite wall, except that the piers were restored to their original dimensions by use of gravity placed 1:2:4 concrete, the whole then covered with the same cement-sand plaster coats, the top one ¾ in. thick. Note the corner angles on the upstream face of the piers in Fig. 10 to provide added scour resistance. The present condition of piers and spillway above the water line is similar to that of the wall mentioned above.

Fig. 13 shows the head gate piers at the intake to this same canal, with the water drawn down, the near piers having been restored, the others not. Note again the corner angles, placed for resistance to scour and to the seasonal conditions shown in Fig. 14. Ice sometimes acts as a cantilever slab extending out from the piers as much as 10 ft., due to lowering of the water after formation of ice in the canal and and the resulting tendency to pull off the concrete pier surfaces gripped by the ice is obvious. The two near restored piers in Fig. 13 were treated exactly as were the spillway piers shown in Fig. 10 and 11, but in the far or third restored pier the plaster finish was omitted, which really qualifies it for inclusion in the "gravity placed concrete" restoration method next described. The cracking and crazing in the former two piers is marked, in the other practically absent.

An effort will be made this spring to determine the quality of the concrete under the plaster, and the bond between the two, in both the crazed and uncrazed areas of these restored structures, by means of the coring tool. Until this is done, no conclusions can be drawn concerning the reasons for the conditions now existing in this covering or concerning the efficiency of this method of restoration given a properly prepared base surface, the elimination of the possibility of water collecting back of the plaster, and proper application and finishing. The above described installations will, however, due to the different conditions applicable to the various units restored, serve as very

FIG. 10—UPSTREAM FACE OF SPILLWAY AFTER RESTORATION

FIG. 11—DOWNSTREAM FACE OF SPILLWAY AFTER RESTORATION (SHOWN BEFORE RESTORATION IN FIG. 4)

satisfactory field trials for the next few years for the method, leading finally, it is hoped, to a fairly definite determination of its applicability.

GRAVITY PLACED CONCRETE (REINFORCED)

As indicated above, the farthest of the three restored piers shown in Fig. 13 was completely treated by this method, which consists simply

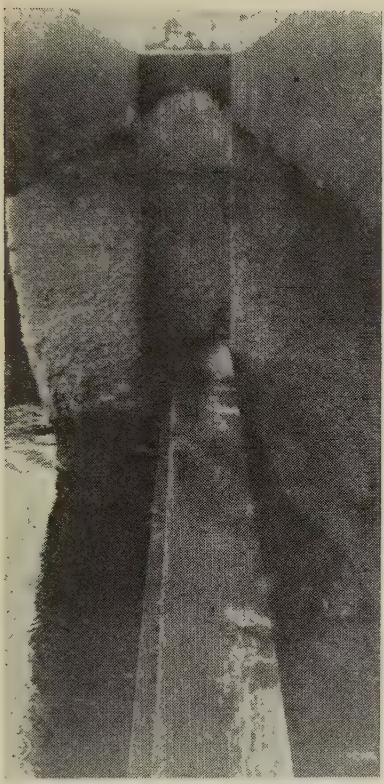

FIG. 12—STOP LOG GROOVE IN NEED OF REPAIRS

of removing all disintegrated material and replacing it with new material reinforced not only as required by the original design or use of the structure, but also to provide adequate integration with the base material.

This latter requirement is not subject to precise determination but its cost is low enough to encourage a liberal use of reinforcing, dowelling, etc. Dowels may be relied on to do their job only if imbedded in the old material to a depth and in such manner as to take into consideration the quality of that material and the stress to which both dowel and material will be subjected.

If the functioning of a dowel is to depend on its bond with the adjoining material, its hole should be big enough for imbedment of the dowel in new grout connecting it with the old material in the wall of the hole. If dependence is placed on the anchorage of the dowel at its end, then a sure means for such anchorage must be provided through the principle of the expansion bolt or the grout embedment of a hook

FIG. 13—HEADGATE PIERS—WATER DRAWN DOWN—NEAR PIERS RESTORED

FIG. 14—ICE ON HEADGATE PIERS

at the dowel end. These dowel requirements merit emphasis—they constitute the key to satisfactory mass restoration work.

Use of this particular restoration method is, paradoxically, not necessarily dictated by a large volume of restoration. It will be used chiefly for small but bulky jobs when pressure-concrete equipment is

not available, or possibly coupled with pressure-concrete to reduce costs where the job includes both thin coverage work and large, bulky units. For it should be remembered that pressure-concrete is really $40-per-c.y. concrete, even in large, bulky units, and gravity placed concrete in sufficient volume and bulk has no difficulty in competing with it.

It should be unnecessary to say—though experience indicates that it isn't—that new restoration concrete should be placed with exact regard for all the well known requirements for good concrete. In addition, the old surface must be utterly cleaned with air or water or both or by other adequate means. A bond cement-sand grout coat intimately applied to the old surface just before placing the new material will help insure complete contact and bond of the new with the old, by filling completely the pores in the latter.

BRUSH-APPLIED, CEMENT-SAND WASHES

An advanced condition of spalling is indicated in the tailrace wall of a power plant shown in Fig. 15—so advanced that reinforcement placed about 6 in. back of the original surface is exposed, particularly in the planes of pour stops. The top crust of concrete just under the water table is removable in large flat pieces, exposing one or two inches of powdery disintegrated material overlying a thickness of 2 in. to 10 in. of dead or inert "concrete" easily removable with air or hand tools. The wall was originally 4 ft. thick at the thinnest point—opposite the outer sweep of the interior face of concrete scroll cases leading to the water wheels. There is no need to point out that this condition was caused by the water from the inside, under head, passing through the wall, freezing, and spalling the concrete at the outer face—the wall wet and dripping all over in mild weather, ice-sheeted in cold weather. There is no need to say that this condition could be remedied in no other way than at the water source—the upstream or water contact face of the structure, although offers were made by some who should know, to solve the problem by treating the downstream face only.

No restoration work whatever has been done on this downstream face—appearances were not important. But every water contact face on the upstream side was restored and waterproofed two and one-half years ago, and since that time the downstream face has been thoroughly dry.

In this case restoration work consisted of cutting away all bad material and replacing it with new. The volumes involved were considerable, especially in the pour joints which were recessed to a maximum depth of 10 in. where the material was bad. Fig. 16 shows

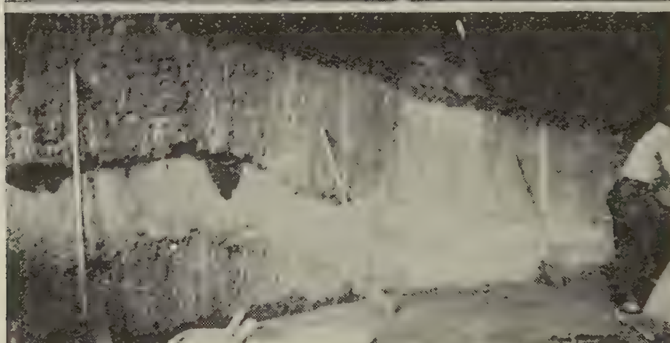

FIG. 15—SPALLING IN ADVANCED STAGES ON TAILRACE WALL

FIG. 16—SCROLL-CASE POUR JOINT FILLED

FIG. 17—WALL FINISHED WITH BRUSH COATS

where the scroll-case pour joint (already filled in this view) runs in a general horizontal plane—the scroll-case floor sloping upward to the rear. In some places, mainly near the water line in the forebay, much material was also removed between pour planes.

After this initial preparation and the usual cleaning with air and water, thin ply wood forms were erected in front of the cavities to be filled, the ply wood held in place by batten and brace forms during the first part of the job, and later, in the interest of economy of both labor and material, by tying the battens back to existing reinforcement or to expansion bolts in the adjacent wall. Pour holes fashioned in the top of each form provided for the easy pouring of a wet 1:1:1 concrete, the coarse aggregate being ¾-in. crushed stone. To offset the shrinkage naturally expected from a wet concrete, 25-lb. of a finely ground iron-ammonia mixture similar to Ironite was added for each bag of cement. After form removal, the entire area of new and old material was evened up, when necessary, by bushing to a fairly smooth surface, ready for the addition of brush coats of grout.

These brush coats were five in number, all mixed to a thick pea soup consistency and applied on the wall by means of a 5-in. medium-stiff bristle hand brush. Each coat was applied not less than 24 hours after the previous coat; the brushing motion was at right angles to that of the next previous coat. These five coats were made up of the following materials:

First Coat—25-lb. iron to 1 bag cement
Second Coat—1 bag cement, 1 c.f. sand, 25-lb. iron
Third Coat—1 bag cement, 1 c.f. sand, 25-lb. iron
Fourth Coat—1 bag cement, 1 c.f. sand, 25-lb. iron
Fifth Coat—1 bag cement, 2 c.f. sand

The last coat contained no iron, and was added for appearance only to cover the previous coat, rust-colored due to oxidation of the iron. Fig. 17 shows a close-up view of a section of this treatment on an area above water, and indicates the fine crazing which developed soon after the work was completed. It is difficult to test for the depth of this crazing since the total thickness of all five coats is not over $\frac{1}{16}$ in. but the core drill previously mentioned will doubtless provide the opportunity to do so. It is hoped and believed that the crazing is confined to the top coat—containing no iron, which is richer than would now be recommended and should have been expected to craze on that account.

The initial iron-cement coat was intended to provide a bond for the ensuing coats, the iron in this and the next three coats, supposedly balancing the natural shrinkage of the cement or cement-sand grout, by expansion due to oxidation of the iron. Its sponsors claim that any

seepage through the top cement-sand finish, would effect further oxidation of the iron content in the adjacent coats and seal any possible crazing therein. Whether or not just that happens is not known but we do have the evidence that this particular job has been successful in its main object—to stop seepage of water from the upstream face through the structure to the downstream face.

Another thing is certain—the installation was expensive. Here was a case where the placing of pressure-concrete in the seams and bad areas (see Fig. 16), thus avoiding all use of expensive forms and poured concrete, and application of the grout coats by hand as described, or even by means of the same pressure-concrete equipment, would have reduced the cost considerably. The cost of the iron was high, and investigation might show that it could be used in smaller quantities, or possibly not at all. At any rate, the cost—approximately \$1.00 per square foot overall—was excessive when compared with a probable overall cost of about 50¢ per square foot by the all pressure-concrete method.

THE "PRESSURE-CONCRETE" METHOD

Concrete "shot from guns" has many characteristics that differentiate it from ordinary concrete. Its placement is unique. Its basic materials, sand and cement, are mixed dry at some distance from the point of intended application, placed in a small metal enclosure under air pressure, and then projected—still dry, by means of this same air pressure, through a hose to an ejection nozzle, where the dry discharge is mixed with an adjacent parallel jet of water. The thus combined matrix is directed at the spot intended to be covered or filled. The new material "grows" on the old as you watch—in the hands of a real expert it grows in the right direction in the right shape and with the desired surface texture. It appears to stick, and does stick when properly applied.

Now the unusual features of "pressure-concrete" have given rise to the generally held idea that almost anything can be accomplished by the method; that there are few, if any, problems of coverage or repair or maintenance, or even new construction, for which it will not provide the solution; and that this may be done without much of the care usually prescribed, where ordinary concrete is involved, in preparation of the base surface, selection of materials, application, curing, maintenance, etc.

Well, it isn't so. Actually some of the difficulties of poured concrete are much reduced in this method, as in the case of placing. Others are almost entirely eliminated—no forms for instance in pressure-

concrete work, no excess water, for its normal water-cement ratio is much lower than the lowest at which concrete can be poured, even when "lubricated." These are real advantages too—they can't be ignored. But there are also difficulties peculiar to this method, some in part and some totally foreign to poured concrete, such as necessity for skilled labor in many of its operations, exacting requirements for mixing, for preparation of the old surface, for placing of reinforcing; a proper finish is not easy to obtain, bond with the base concrete is critical, aggregate must be unusually clean, hard, sharp; pressure, angle, distance, thickness, etc., of application, all affect the finished job; subsequent spalling, cracking, crazing, are all too frequent—let's not skip these features either. I am convinced there is no other method so flexible and convenient in use, so economical in its particular field, so permanent when properly done, as pressure-concrete—nor any so readily misused or misapplied, or so frequently misunderstood. No advantage to be gained by ignoring this, and much is to be gained by conceding it—and then doing something about it.

Now I propose in the discussion of this particular method, just as in the more inclusive subject of general restoration, to list the conditions under which it is applicable, the manner in which it is currently accomplished, the results now obtained, some of the reasons why these are not all that is desired, and finally, some suggestions for improving these results.

THE PROPER BASE

Pressure-concrete is peculiarly advantageous for use in thin sections, where forms would otherwise represent the larger part of the restoration cost. It is doubtful if such sections may normally be less than 1½ in. thick. It may be applied on almost any concrete surface provided, naturally enough, that the surface has been properly prepared by removal of all unsound material. Permanent bond of the new with the old is essential, but is unthinkable unless the old is sound. Which is not to say that the old material must be as solid and as sound as modern high strength concrete; but it most certainly cannot be spongy, loose, disintegrated. Where the old material ceases to be unfit, and begins to be fit for the application of pressure-concrete, can be decided only by those whose judgment has been schooled by experience—experience which will embrace pressure-concrete spalled from base concrete which was not good enough, as well as pressure-concrete which sticks and does not crack, and defies the passage of time, extremes of weather, and other local conditions. These experiences of spalled or sound pressure-concrete met with in the same structure too—

FIG. 18, 19, 20—STAGES IN RESTORATION WITH "PRESSURE CONCRETE"

in adjacent sections of the same structure; which indicates that in pressure-concrete particularly, the price of success is eternal care—care in preparation of the base, care in selection of materials, care in mixing, placing, finishing, curing.

Fig. 18 and 19 show about the same section of a concrete structure respectively before removal of disintegration and when finally ready for application of pressure-concrete. There is very little difference in the *appearance* of a surface ready for pressure-concrete and one not ready—and that is an important point. Appearance frequently is no guide at all to the satisfactoriness of a base for the application of restoration materials. As previously mentioned, the sound of a material under the blow of a hammer is the preferred and indeed, the only reliable guide.

There will be encountered at times, however, in restoration work, concrete structures which defy all effort to uncover this ideal sound base material. Sometimes the unsoundness is not the result of a surface breakdown due to seepage, freezing, thawing, etc., but rather of a basic lack of quality in the entire body of the structure. Demolition of the structure or its structural impairment might readily result from a strict adherence to the requirement for a "sound" base. In such cases, something short of this requisite must be accepted and other steps taken to insure adherence of the new material to the old.

The steel mesh used in pressure-concrete is not a bond requirement. Its lack of value in that respect is obvious when the 30-in. spacing of anchors attaching the mesh to the concrete is considered. The mesh merely gives continuity to the pressure-concrete covering, reduces the magnitude and increases the number of shrinkage cracks. If the normal bond between the pressure-concrete and the base material may not be anticipated in any given case, a satisfactory structural bond may be provided by installation of a reinforcing bar mat in the pressure-concrete zone, this mat to be integrated by welding at its joints, and the whole attached to the base concrete by welding it to dowels in-inserted in the base to such length, in such number, and in such manner as to preclude the possibility of their withdrawal. The size and spacing of both bars and dowels, and the length of the latter will be admittedly arbitrary, as the forces they are to balance are not determinable Fig. 20 shows a close-up of a spillway crest with disintegrated material removed, where an added bond was thought necessary due to unusual conditions, and where bar reinforcement was replaced by additional layers of standard mesh.

Some consecutive steps in restoration work interfere with each other, especially in restricted areas. A completely cleaned and

meshed area may be covered with a coating of "rebound" (aggregate which fails to adhere to the base concrete or pressure-concrete matrix and falls away from the restoration surface) from an adjoining area being "shot." Deep chipping operations will deposit dust on adjoining areas ready for pressure-concrete. If these layers are not removed, they will constitute a positive preventive to "bond," and it is impossible therefore to over-stress the necessity of thoroughly cleaning these surfaces with available means of air or water before actual "shooting" begins.

MESH REINFORCEMENT

Since the mesh used in pressure-concrete is for shrinkage abatement or deconcentration, its material, wire gage, mesh measurement, and bond characteristics should be correlated with its enveloping medium—pressure-concrete. 3 in. x 3 in. x No. 10 x No. 10 or 4 in. x 4 in. x No. 7 x No. 7 galvanized steel electric-welded wire meshes are ordinarily used, but an experimental study of their adequacy for the purpose would not be out of order.

Ordinarily in a pressure-concrete coverage of 1½ in., the mesh is placed with the wire centers ½ in. from the base surface, thus providing a 1-in. coverage of mesh, as measured from the center of the mesh wire. But one contention that has been made for pressure-concrete is that any crazing or cracking in its surface cannot go deeper into the pressure-concrete than the mesh. If this is so, and obviously it should be desirable to keep these cracks as shallow as possible, why not place the mesh nearer the surface? Say in the middle of the pressure-concrete zone, or possibly closer to the surface than that. Another subject for study.

In any event, wherever the mesh is placed, a positive method of so placing it is imperative. It's easy too—but you won't get it unless you go after it. If the contractor has complete freedom of action, your mesh will almost always be quite tight against the base, because it's easier to bend the expansion bolts that hold the mesh in place over tight against the mesh, in case their holes haven't been drilled deep enough into the concrete, than to be careful to drill the holes to proper depth in the first place, for holding the mesh in its proper place. And what good is the mesh against the base concrete? In that position it doesn't serve its fundamental purpose for it isn't even imbedded in the pressure-concrete, and further, it reduces the area of contact of the latter with the base, and therefore the total bond. Yet a simply made and simply applied "wire chair" placed under the mesh will locate the mesh properly. Insist on its use!

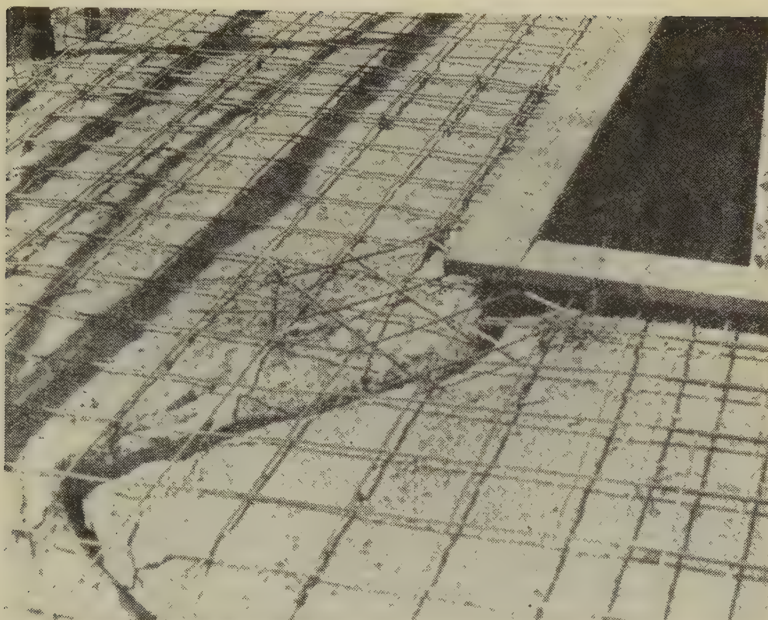

FIG. 21—RESTORATION AT ANGLE WITH KEYED CONSTRUCTION JOINT

Experience shows that many of the cracks in pressure-concrete are developed at the vertices of interior angles in the structure, due primarily to the varying expansion and contraction conditions set up in adjoining sections of the structure of different mass characteristics. If the cause of these is correctly inferred, they may be eliminated or at least reduced in extent and size by additional reinforcement. Fig. 21 is a top view of a large masonry structure at an angle in that structure (showing prominently an original keyed construction joint), with disintegrated material removed and mesh applied ready for application of pressure-concrete. Note the added mesh at the corner to reinforce against cracking.

CRACKS AND JOINTS

What about the joints or major cracks in the base concrete? Two kinds generally—horizontal and vertical. The former are usually the result of bad original workmanship or material. They can be cut out and filled as in the case of any other bad spot. No subsequent cracking through the pressure-concrete because there's no movement. But the vertical cracks are not the same. No need to determine *why* they're there. They're there, that's certain—either put there in the construc-

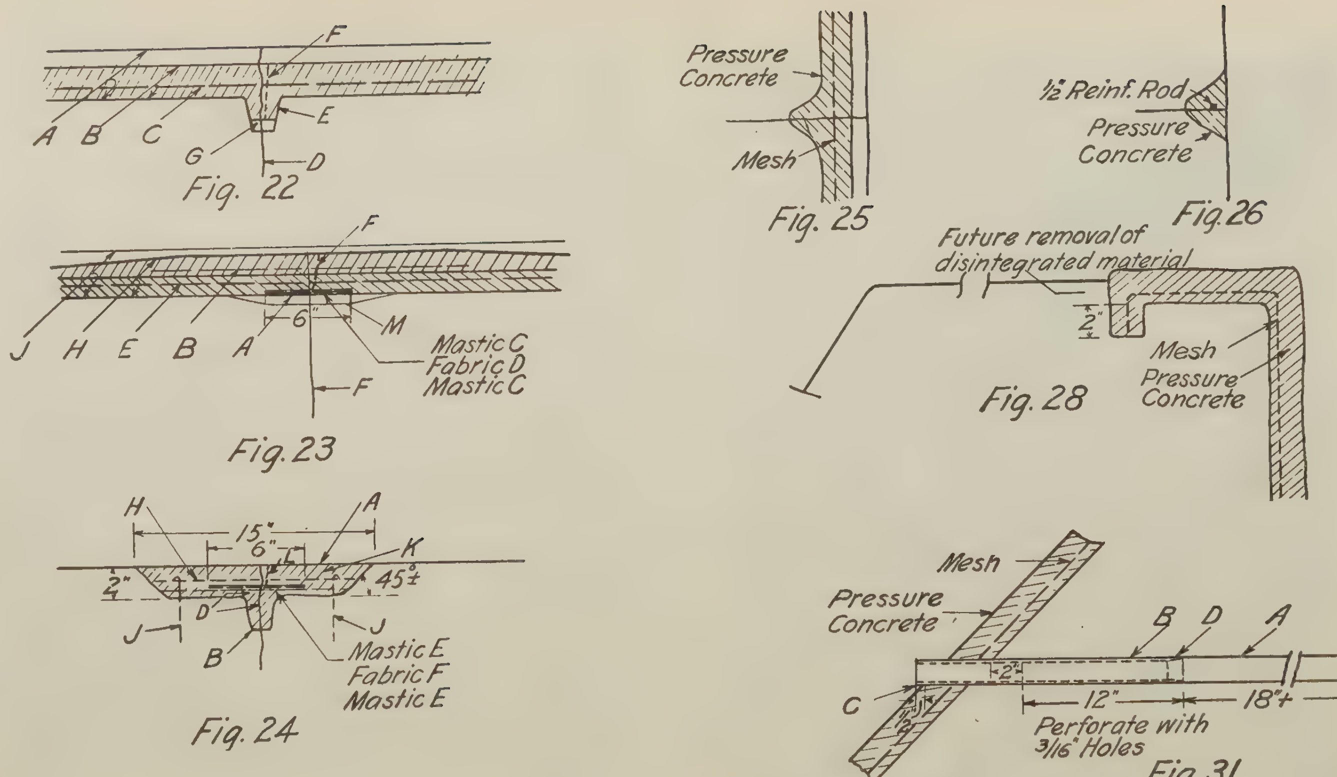

FIG. 22, 23, 24 (LEFT) AND FIG. 25, 26 AND 28 (RIGHT)—JOINT AND CRACK DETAILS—SEE TEXT
FIG. 31—DETAIL OF WEEP

tion operations as expansion joints, or to sectionalize the work, or developed after construction due to expansion and contraction or to movement of some other kind. And if they are in the base concrete they'll be reproduced in the restoration work. No use ignoring it—they'll be there. The task isn't to prevent the occurrence but rather to anticipate it. The cracks in the pressure-concrete do no harm in themselves but they do let moisture in, and then the same old story of disintegration. If the correct basis of restoration work is in general one of waterproofing, as heretofore implied, you'll have to keep the water out of these cracks, and to do that the preventive must be applied during the restoration work, not after.

Fig. 22, details one type of restoration joint on the upstream face of a water impounding structure, which joint I understand is patented. Disintegrated material (A) on the face of the structure is removed in readiness for application of pressure-concrete (B) with standard reinforcing mesh (C). A trapezoidal section (E) about 2½ in. deep, is scored out of the crack or joint (D) with the base centered on the crack and about 1 in. wide. Into the base of this groove is placed a suitable mastic material (G) such that it will not run under summer conditions, nor brittlize under winter conditions. The remainder of the groove is filled with pressure-concrete at the same time as the main restoration work (B) is done. Subsequently a crack (F) will develop in the layer (B) over or near crack (D); water will enter from the upstream face but will be prevented from reaching the crack (D) by the mastic (G). Cutting of the mesh (C) at point (H) opposite the crack (D) tends to locate the crack (F) over the original crack (D). This joint is undoubtedly effective if properly applied in the field, and that *can* be done, but there again it won't be unless properly and continuously supervised by someone with authority outside of the contractor's organization.

Another joint is shown in Fig. 23. It has the advantage of being easier to apply satisfactorily in the field. The vertical crack (F) is covered by the pressure-concrete pad (A) this latter acting merely as a base for the application of coats of mastic (C) and waterproofing fabric (D) which follow. If the surface (M) is bushed down smooth the pad (A) can be eliminated. After the disintegrated material has all been removed, and pad (A) placed over the crack (F), a layer of pitch or mastic (C) is brushed or trowelled thereon to a thickness of at least ⅛ in. and to a width of 8 in. Centered over the crack (F) in this mastic is placed an 8-in. wide strip of waterproofing fabric (D), and over this is brushed or trowelled an additional ⅛-in. thick layer of mastic or pitch (C). Over the entire joint, and ½ in. therefrom, are placed sections of mesh (B) for reinforcing those portions of the subsequent pressure-concrete layer which will cantilever over the mastic joint. The standard mesh (E) is then placed over the entire structure, both meshes are clipped over the joint, and the pressure-concrete (H) shot to the required depth. Subsequently the pressure-concrete (H) will develop a crack (G) over or near the original crack (F) but the waterproofing effect of the mastic (C) and fabric (D) will prevent moisture from gaining access to crack (F) in the original wall. Here again eternal vigilance during construction is the price of an adequate job.

Details of a vertical restoration joint to be used where the joint requires restoration but the adjoining surfaces do not, are shown in Fig. 24. A wide, shallow groove (A), with sloping edges (L) is cut out so as to center over crack (C). Notch (B) also

centered over crack (C) is then cut out and pressure-concrete (D) is shot in as shown to fill notch (B) and provide a smooth pad for the application of mastic (E), fabric (F), and mastic (G) to a width of 6 in. Standard mesh (H) is then placed across the groove (A), ½ in. from mastic (G) and fastened by means of bolts (J) on each side. Finally, the remainder of groove (A) is filled with pressure-concrete to the original surface.

Fig. 25 is a detail of the restoration of a horizontal joint where the entire area is restored, and Fig. 26 where only the joint or crack is restored.

There are other currently used restoration joints, involving the use of metal expansion plates, etc., but for the type of work herein contemplated, these are too expensive, too difficult of execution, too problematical as to effectiveness, and in some cases, likely to do more harm than good to the adjacent pressure-concrete.

In downstream restoration work the vertical joints in the base concrete will again be reproduced in the restoration work, but in this case they are subjected to seepage only from overflow water or precipitation, and may in general be ignored since the open joint has the counter advantage of acting as a weep for seepage of possible entrained moisture in the heart of the structure.

CONSTRUCTION JOINTS

There are current two methods of making construction joints in pressure-concrete. One in which the work is always completed in any one period of operation up to a "screed strip" placed for that purpose, providing a square face against which the next adjoining pressure-concrete is shot. The other, in which the pressure-concrete material at the end of a period of operation is thinned out, in a distance of about 12 in. to a feather edge, and at the next operation this wedge piece is covered by an overlapping wedge of similar width.

Fig. 27 is a view of the restored face of a spillway in which the square horizontal joint was used. A slight efflorescence marks this joint in some places and a crack is easily discernible in practically the entire length of this joint, indicating that a tight joint between the adjoining areas was not obtained—and in fact, would have been difficult to obtain. I have never seen such a construction joint-mark in any case where the feather edge joint was used, and the only possible conclusion favoring the feather edge joint is obvious.

END JOINTS

In view of frequent need for restricting restoration work to the upstream face of water impounding structures, there will always be the necessity for a proper detail of the ending of the restoration work. It will not suffice merely to taper off the new pressure-concrete to an indefinite line, for disintegration of the base concrete will continue

FIG. 27—SQUARE HORIZONTAL JOINT

under the taper and a raw cantilever edge of pressure-concrete will result, defying continuance of the restoration work at a later date without considerable preliminary work.

Fig. 28 is a detail of a suggested end joint, applicable to either a horizontal or vertical face, and Fig. 29 shows such a joint on a horizontal face before and after shooting. Note the turn-down of the mesh into the sound base concrete to insure no undermining of the new pressure-concrete. Future restoration work can be continued adjacent to this joint without disturbing it in any way. Such a joint would not be desirable at the top edge of restoration work on part of a vertical face. Nor would any other joint, as obviously any such face should be completely restored, at least to a top horizontal plane, and preferably across that plane and down a short distance on the other side.

MIX

Mixes of pressure-concrete vary from 1:3 to 1:6, aggregate measurements being assumed at the mixing board. Since the "rebound" of aggregate during the shooting process is considerable, the actual mix of pressure-concrete in place is richer than the mixing board mix. That is, 1:2½ pressure-concrete will result from a 1:3 mixing board mix, and 1:4½ or 5 from a 1:6 mixing board mix. Crazing and cracking will increase if the mix is too rich and experience indicates that a mixing board mix of 1:4 will yield the best results.

AGGREGATE

Any clean sharp sand, not too fine and passing a ¼-in. screen, can be used for pressure-concrete work. There is presumably a sand ideally

FIG. 29—END JOINT AS DETAILED IN FIG. 28

graded for the purpose, but it would be difficult to obtain agreement as to what that ideal grading should be, difficult to obtain it economically in practice, and difficult to ascertain the effect of rebound on the grading in the finished pressure-concrete. Specifications calling for sand passing the requirements of the New York State Highway Department, known to be rigid as to cleanness and grading, have proved satisfactory. Within such restriction, the contractor should be permitted full latitude in the selection of a sand which is satisfactory to him for shooting purposes.

AT THE NOZZLE

Much depends on the actual application of the pressure-concrete. A good nozzle operator means much, particularly in respect to the proportion of water used, the angle of discharge on the surface being restored, the distance the nozzle is held from that surface. The distribution or build-up of the restoration material on the surface, the prevention of coverage of pressure-concrete on collections of "rebound," the complete filling of all openings and corners in the base, cleaning the base of all dust or dirt—these are matters which depend primarily on the nozzleman, and the most intense supervision over a mediocre workman will not suffice in exchange for a nozzleman with a long and successful apprenticeship in this type of work.

As in concrete, the less water used—assuming enough for hydration of the cement—the better, although again as in concrete, we find the nozzleman tending toward excessive use of water for easier placement. The distance the nozzle is held from the surface being restored affects the work markedly. The discharge loses velocity the farther out it cones, and the compactness, impermeability and strength of the pressure concrete are proportional to the striking, and not the nozzle or pressure-machine velocity. Yet pressure-concrete can be placed uniformly only if its spray cone has spread to a fairly large cross-section, the best effective range or distance from a flat restoration face being apparently about 36 in., to be increased or reduced as required in restricted or difficult-to-get-at places.

AT THE PRESSURE MACHINE

But the nozzleman is limited in what he can do by what he gets from the pressure-concrete machine, his control of the matrix being confined to the amount of water used, adjusted by means of a valve just back of the nozzle. A velocity of 300 ft. per second at the nozzle for the aggregate, and a pressure of 50 p. s. i. for the water at the nozzle are believed to produce the best pressure-concrete. A device for measuring this velocity, and a pressure gage for measuring the pressure are

available for installation at the pressure-concrete machine, but these are of little avail unless consulted continuously for correction. And even then are of little value unless the nozzleman maintains at the nozzle the conditions which they assume.

LAYER THICKNESSES

Specifications should call for a maximum layer of 2-in. of pressure-concrete to be applied on a vertical surface, and 3 in. on a horizontal or sloping surface. There is, however, good reason for assuming that any overall thickness of pressure-concrete will be more compact, more homogeneous, and yield a smoother top surface if placed in two or even three layers, each layer allowed to attain its initial set before the addition of the next layer, and the top of each layer adequately cleaned before the next application. Patently the advantage in discharging pressure-concrete material at high velocity on a surface assumes that surface to be rigid, yet rigidity of that surface becomes progressively less as the pressure-concrete layer increases in thickness. Hence, the obvious desirability of applying two or three thin layers rather than one thick one. A smoother top or finish surface will also be easier to obtain since a rough wavy surface is the inevitable outcome of having to place the final portion of a single thick layer on the non-rigid base formed by the early portion.

A field trial was made to see if the multiple layer idea would make any difference in the smoothness of finish, and also as a test for bond between the layers. The first was a difficult matter to determine in a small area for the nozzleman knew what was going on and this knowledge would naturally affect the result. A more extended trial in that direction should and will be made. The matter of bond will be checked by obtaining cores. If the results of these tests justify the present assumption of the superiority of the multiple layer method, it should find itself readily enough in pressure-concrete specifications.

SURFACE FINISH

Fig. 30 is a close-up of an untouched area of single layer pressure-concrete. Note the pitting due to the larger pieces in the aggregate, and the surface sheen. This brings up the question of the finish surface. There is rarely a doubt in anyone's mind that the exterior surface of *any* masonry structure is a really vital part of that structure from a maintenance or life point of view. The entire solution of restoration problems is premised on that assumption. We have all seen many concrete structures ruined by the innocent enough process of spading in the forms, or trowelling after removal of the forms, or by the definitely inane bushing of the surface after the permanent set, to

simulate a rough stone finish. These things aren't done so much now because of the lessons their doing has taught in the past.

Now we have again with us the problem of "surface finish"—this time with respect to pressure-concrete. As previously stated, the type of structure herein chiefly referred to is generally located where the casual observer does not see it, and there is no great need to consider its detailed appearance. Granted that if a beautiful pressure-concrete exterior could be obtained at no extra cost, and without sacrifice of quality—why not have it? But you can't. It will cost more and it will *not* be as satisfactory. Imagine if you can the pathos in a contractor's superintendent's plea to be permitted to "touch up" the surface of a pressure-concrete wall so that it will not look so rough! Pride in his work—yes. But a superficial pride that should mean nothing to the owner, for it involves scraping off, by means of trowel or screed, the surface of the pressure-concrete when still green, but after its initial set; and then the addition of a "flash" coat of pressure-concrete to cover the scars of the scraping operation. And the valuable armor of the natural finish of an untouched surface, as shown in Fig. 30, is lost in just the place where it is needed the most—where it is not covered by water and hence is subject to all the punishment the elements can mete out. Don't let them do it! Write specifications to provide that all pressure-concrete work will be brought to a smooth finish *in* the shooting process, and then be left alone. Exceptions must be made in some cases, of course, when the resulting work must be left to a true contact or level surface—but they should be real exceptions.

CURING

The necessity for curing pressure-concrete is as definite as in any concrete. But since pressure-concrete generally covers very large areas, it is not so simple. The best way of curing pressure-concrete is to maintain it in a continuously wet condition for the curing period, or until covered by more pressure-concrete. This period should be specified as four days, which is doubtless sufficient if really continuous.

Here again the simplest part is to include it in the specifications and the most difficult to see that it is actually done. Every capable superintendent knows the value of curing; knows its necessity; is or should be familiar with the fact that lack of it may ruin an otherwise good job. But the urgency of other phases of the work and a natural inclination to overlook details combine to nullify the insistence of the owner on this ridiculously inexpensive insurance against job failure. And again the answer is—*insist* on it. In large vertical areas a perforated water pipe can be rigged up to discharge over the edge of the work or the

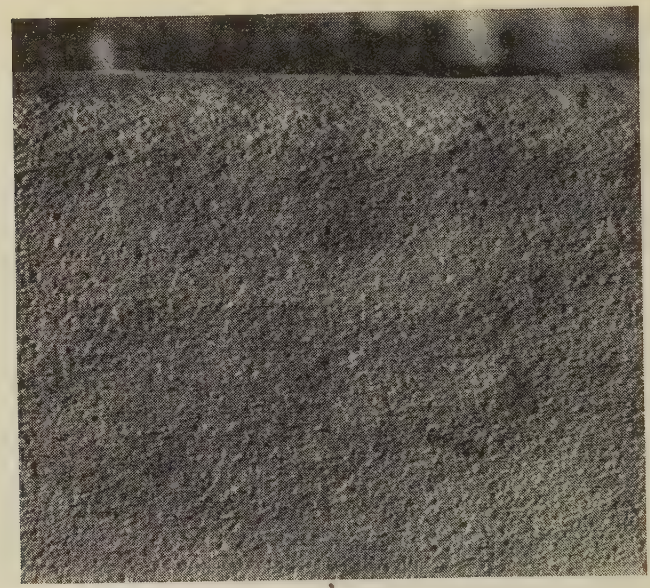

FIG. 30—PRESSURE CONCRETE SURFACE

areas being cured can be continuously wet down by hosing, but by whatever method accomplished, insist on a continuously wet surface during the curing period.

WEEPS

The detail of a weep in pressure-concrete on a downstream face is important. Its purpose is to prevent impounding of water back of the plane between the original concrete and the pressure-concrete. Weeps are like dowels—they look all right after they are in, but their efficacy depends entirely on the manner in which they were installed, and that cannot be determined after the work is completed. The desirable length, direction and size of a given weep is a very difficult thing to determine in advance. Where it is intended to drain a definite flow of water, the task is easy. But a general seepage must be taken care of by a regular spacing of weeps over the area involved, with the idea that additional weeps may be placed where needed after the pressure-concrete work is completed, and the need is then made apparent by wet surface spots.

Fig. 31 is a detail of a satisfactory weep to be placed either before or after pressure-concreting work is done. The drilled hole (A) has a minimum diameter of 2 in., minimum depth 24 in. in the base concrete. A galvanized iron pipe (B) has a minimum depth in base concrete of

12 in. but not less than twice the length protruding there from. Pipe (B) should be perforated freely with $\frac{3}{16}$ in. holes from its inner end to within 2 in. of its emergence from the base concrete, to allow for seepage entrance. The inside pipe bevel (D), 1 in. long, will reduce ponding back of the end of the pipe, and will also facilitate cleaning. Overhang (C) should be about ½ in.—long enough to provide a drip and yet not enough for ice or debris to be caught on it, its outer end cut on the same slope as the surface to provide as little obstruction as possible.

The weakness of weeps in pressure-concrete is in the possibility of carrying seepage to the plane between the base concrete and the pressure-concrete. This is avoided in the weep shown in Fig. 32 by making the metal pipe a tight fit in its hole, and making sure that the unperforated portion laps over the pressure-concrete base-concrete joint.

Invariably weeps are placed and then forgotten. But they are in the nature of safety valves to preclude the possibility of building up impounded water heads, and as such should receive regular inspection for cleaning purposes.

INTEGRAL WATERPROOFING

Some reference has been made to the integral mixing of certain fabricated materials in pressure-concrete to overcome the crazing and cracking difficulties. In the description of the "cement-wash" method of restoration and waterproofing, reference was made to the use of an "iron" admixture for counteracting shrinkage and consequent cracking and crazing. The job of waterproofing the surface treated was there effective, but it is not known, as stated before, whether or not the iron played an essential part in this success. Extended experimental work would be required to make this point clear.

During the construction seasons of 1934 and 1935, the addition of waterproofing materials was made in certain selected areas adjoining areas in which these materials were not used. There are, therefore, available for observation what amount to field laboratories for determining whether or not these materials have actually accomplished the purpose intended.

Now the efficacy of these included materials need not be immediately discernible to be real, in spite of their sponsors' claims. It is possible to visualize a set of conditions wherein adjoining areas, one in which the admixtures have been used, and one in which they have not been used, will show no difference in their condition for the first few years after installation, and yet may show a marked difference after a lapse of time. To date the adjoining areas referred to show absolutely no

difference. Each has its share of crazing and of cracking. It may be definitely said that their inclusion does *not* eliminate these feature—in fact has absolutely no effect on them as far as can now be seen. The conditions applicable to each are as nearly identical as could be on the same structure, and in adjoining areas on that structure. Yet it cannot be said, on the other hand, that they have no value for this may be brought out only by time. Meanwhile it appears obvious that the large expense involved in such inclusion is not justified, except on such a restricted experimental basis.

GUARANTEE

Much can be said on both sides of any discussion concerning guarantees. The statement often advanced that a guarantee is not better than the guarantor—that it is of no value from an unreliable guarantor and unnecessary from a reliable one, is trite. That may be true, but human nature being as it is, there is no gainsaying the fact that a guarantee is the source of much confidence in any transaction, for it has the three-fold effect of—

1. Summarizing the results to be expected with particular relation to time.
2. Eliminating at least some of the possible unreliable prospective bidders, and
3. Emphasizing to the successful reliable bidder the hazards of a poor job from whatever cause, and the necessity for bending every effort to avoid its consequences.

Guarantees in connection with pressure-concrete work have all the vulnerable characteristics of other guarantees, plus some resulting from the fact that job failures may be attributed to a greater variety of undeterminable or uncontrollable causes. Yet if companioned by a reasonable amount of freedom to the contractor in the solution of detail problems arising in the field, their value to both owner and contractor is real. A five year guarantee appears to be fair in view of the present status of the pressure-concrete industry, and the reasonable assumption that this period should usually be sufficient to disclose poor workmanship, poor materials, or poor methods.

ANTICIPATED RESULTS

An owner should not be adjudged unreasonable if he anticipates his restoration work will have, say, a 20-year life, during which little or no maintenance would be necessary. Lacking assurance of such a minor-maintenance period, it is doubtful if the general policy of restoration as applied to lengthening the life of concrete structures could be justified. This is not to say that no defects in the restored material would appear—no cracks—no crazing. But by the ordinary current methods of maintenance, the work should persist without the necessity of replacement for at least 20 years.

Cracks and crazing are not of themselves an indication of failure in pressure-concrete. If the former go no further than the reinforcing

mesh, and the latter are superficial, there can be no justifiable complaint. But if the cracking goes through the pressure-concrete layer, so that moisture will get to the base concrete, and so lead inevitably to freezing and loosening of the pressure-concrete from its base, or if the crazing grows into a spalling condition, either of which possibilities might be attributed to poor workmanship or material, we certainly have a job which does not come up to normal, reasonable, expectations.

No job can be expected to be 100 per cent perfect. A guarantee at once implies such imperfection, and provides the means for correcting the faulty parts. No matter how painstaking a contractor may be, the human element enters into all his operations and always tends to lower the finished job below perfection. But the defects encountered should be individual and not general—they should be limited in scope and rare enough to be the exception and not the rule.

Interpretation of the causes of the many and various types of cracks and crazing in pressure-concrete is indeed a difficult task. Careful inspection, study and analysis over a long period will be necessary to arrive at conclusions in this regard. Until then it will be difficult to guard against any but the most obvious causes. One major difficulty underlying such determinations is the fact that most of the evidence which would lead to them is covered up by the work itself. One way to retain this evidence is by photographing all the areas involved, before restoration work begins, after mesh has been applied, and after the work is completed. This will preserve for later study a record of the conditions prevailing at each step in the restoration work and may readily lead to rational conclusions as to the causes of many of the known defects of pressure-concrete.

There is much to be learned concerning the pressure-concrete method of concrete restoration work. Let the industry take cognizance of that fact, determine definitely its restrictions in use, its basic and detailed difficulties and failures, their causes; and from such data develop definite and sure, rather than haphazard, specifications for a continued but vastly increased use.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.

Concrete Maintenance*

L. F. HARZA† AND H. G. ROBY‡

During the last twenty years the practice of concrete construction has progressed from the stage where it required only a strong back and weak mind, to the basis of a highly skilled art, if not indeed approaching that of a science. The appropriations for concrete research have probably not been exceeded by those for any other purpose. Most of this study has been directed toward improvement in the manufacture of new concrete. With this accumulated knowledge, good concrete is reasonably assured but only by the exercise of that greatest human quality of "eternal vigilance" in every step from the selection of the materials to the curing of the final product.

During the last few years the manufacture of concrete for new work, except government work, has proceeded at a very slow pace, and financial conditions require the maximum preservation of the structures we already have in order to assure them the longest useful life.

The instances of quality failure of concrete built before our present knowledge of the art was available are so many, and the total investment in crumbling, disintegrating concrete represents so vast a sum, that we must recognize not only the problem of building good new concrete but also, as a major engineering problem of equal importance, that of salvaging old structures. We must learn how to restore or repair old defective concrete so that disintegration can be arrested and so that these investments need not be written off prematurely as a loss. This problem has not as yet received its fair share of attention from engineers and research laboratories. In fact, it has not, in general, been recognized by the owners as an engineering problem.

RECONDITIONING AN ENGINEERING PROBLEM

This field of activity has been largely one for patentees or companies developing special equipment for restoring disintegrated surfaces and patentees of special materials for bonding the new coat to the old concrete and for waterproofing the old surface before applying

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936, by Mr. Roby.
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the new. The art is now in a stage of evolution similar to that once experienced by water filtration. For many years, engineers were not called upon to specify and design water purification and filtration plants, but competitive bids were invited from companies controlling various filters and filter processes. Likewise, the work of reconditioning, or restoring, defective concrete is not treated as an engineering but rather as a contracting problem. One must be equipped to handle the construction work against competitive bids in which a part of the sales appeal is some bonding, waterproofing or hardening compound or controlled equipment process. The concrete engineer as such is seldom consulted.

A search of technical literature reveals the results of but little research or of open discussion to determine the merits of the various proprietary remedies for the diseases to which poorly constructed concrete is subject. Independent research laboratories with due discretion seldom investigate or pass upon proprietary products or processes. The information available concerning these processes or materials is often not complete and is seldom of the type developed in scientific laboratories.

The art cannot advance in proportion to its importance without research and open discussion. The several proprietary remedies are for the most part simple chemical compounds or mixtures well known to experienced concrete engineers under their true chemical names. These remedies are not all bad or even indifferent; in fact, some are known to be even good in their proper application and some companies are undoubtedly doing meritorious work. But the problem is vastly greater than special admixtures, adhesives, waterproofing materials or equipment. It is an engineering problem of the first magnitude, one that is now in a very unsatisfactory state because of conflicting claims and lack of proven merits which would be revealed by published research. The situation calls for a vast amount of study to save from premature decay many millions of dollars worth of incorrectly built concrete.

The danger and tendency in the present situation is that a given material or process will be urged for all conditions whether applicable or not, whereas the professional concrete engineer is at liberty to recommend any material, process or equipment best suited to special conditions. For his purpose there should be recognized standard specifications defining the essential requirements for applying a new surface coat on old concrete, for testing the bonding or waterproofing value of materials and any special workmanship details offered in competition.

The specifications should be broad enough, until knowledge is more general, to admit to competition all proprietary equipment, materials or technique of application, subject to test and demonstration of merit before application. Purveyors of products of merit would have nothing whatever to fear from such procedure. The good would survive, and in time become established and their business improved; the poor would fall by the wayside, which they should do.

The authors' interests are more directly concerned with problems of dams and hydraulic structures, probably the most susceptible to attack by seepage and frost, and the most difficult of any structures to repair. During the season of 1932, one of the authors was interested in the repair of disintegrated concrete of a large dam in a northern state, the preliminary studies and negotiations for which revealed the state of the art as has been described. It is believed that a description of this repair and reasons for selection of adopted methods will be of interest to concrete engineers and especially those concerned with masonry dams.

DISINTEGRATION OF CONCRETE DAM

The dam was built in 1922 of a natural gravel without regrading. Ample tests at the time indicated absence of organic matter and satisfactory strength of compression specimens of specified mixture from stratum selected for use in concrete. The dam was built by the owner's own forces and no engineering inspection of the work was called for by the owner, although a consulting engineer was employed to prepare the design. The history indicates that every principle and practice essential for making good concrete was violated, including carelessness in stripping of clay over-burden from gravel pit, use of excessive mixing water, "hydraulic sluicing" of concrete in flat spouts and in the forms to save shovelling or moving of spouts for proper distribution, continuous piling of concrete in huge conical piles from fixed spouts, the thin materials washing to the distant parts of the forms, the gravel remaining in a central cone.

Ten years after construction, repair became essential. The surface of the ogee spillway had disintegrated and washed away for a depth of several inches. Construction joints were visible by laitance seams throughout the structure. In some places concrete could be dug with a pick. The gate piers required complete removal and rebuilding. In the course of removing the old piers the concrete was found to be relatively sound above the elevation of the top of the gates, requiring air tools; but below this elevation removal work progressed three or four times as fast. The conclusion is apparent that the pier concrete

FIG. 1 (LEFT, ABOVE)—DISINTEGRATION OF CONCRETE ON DOWN-STREAM WALL OF POWER HOUSE

FIG. 2 (RIGHT, ABOVE)—DISTANT PIER DISINTEGRATED; NEAR PIER REPAIRED

FIG. 3 (LEFT, BELOW)—DISINTEGRATION OF GATE PIERS AND SPILLWAY

FIG. 4 (RIGHT, BELOW)—DISINTEGRATED CONCRETE REMOVED FROM SPILLWAY AND DOWELS PLACED READY FOR NEW CONCRETE

below pool level, where continuously saturated, had been ruined either through solution by seepage water, or by freezing, or jointly. Concrete upstream of the gate sills seemed to be in fair condition.

Concrete in a dam in cold climate is sure to be subjected to a freezing temperature to great depth on the downstream side often nearly through the dam. Whether a concrete can stand the expansive force of water in its pores depends upon the tensile strength of the concrete and the thickness of the walls surrounding each pore. A pipe with thick walls will not burst by water freezing within it, while a thinner walled pipe will burst. The thickness of the pore walls is an inverse function of the porosity.

The Portland Cement Association has demonstrated amply that disintegration by freezing progresses from the surface inward. The residual concrete, if originally of good quality, can withstand many cycles of alternate freezing and thawing temperatures, even when fully saturated, before its strength is seriously impaired. The theory which seems to explain this fact is that only the water in the pores close to the surface actually freezes because the strength of the walls surrounding the deeper pores prevents the expansion accompanying freezing.

The repair of any concrete subjected to water pressure, should, if possible, be made by placing a new impervious and waterproof surface on the upstream side. Such procedure was impracticable in this case without drawing the water down, thus interrupting operation for several months for cofferdamming and repairing. Even if such a repair were made it would not eliminate the need of a heavy new face of concrete on the downstream spillway surfaces, to arrest disintegration. Extensive grouting through closely spaced holes near the upstream face was suggested for filling the seams and joints, but it was not thought the cement would penetrate the large mass of porous concrete sufficiently to produce the necessary water-tightness unless the holes were very closely spaced.

None of these methods would obviate the absolute necessity of repairing the downstream face of the dam against further erosion and weathering. If in this repair the downstream face could be made impervious and thus produce a water-tight structure, it was hoped that the necessity and tremendous expense and interruption to operation for a new upstream face could be saved. At least no unnecessary work would have been done to try this method.

REPAIR MEASURES

To make a proper repair, this new downstream facing must be: 1—Essentially water-tight and as dense as possible; 2—Anchored to the

old concrete sufficiently to withstand full headwater pressure; 3—Bonded to the old concrete, after cutting off all disintegrated material, so well that only individual pores and no extensive area of separation will ever form at the contact; 4—Placed upon a surface of sufficient roughness or undulating contour to prevent the possibility of "creeping" and consequent separation from the old surface as the result of shrinkage and temperature changes; 5—Thick enough to have strength and rigidity in itself and to minimize the number of cycles of freezing and thawing which reach to the depth of the old concrete; 6—Reinforced to span between anchor rods with negligible deflection.

One-inch round anchors were grouted at least 40 inches into the old concrete on variable spacing and designed to develop strength to resist full headwater pressure in a sheet behind the fâcing, should such an unlooked for emergency develop. It should also be observed that the maximum head of 50 feet could produce only about 23 pounds per square inch of tension between old and new concrete which a carefully bonded joint should readily resist even without dowels.

Bonding was accomplished by first chipping out, with air chisels, all weak and defective concrete amounting to perhaps 8 in. in average depth. The surface was sandblasted just before placing concrete to remove organic matter which formed subsequent to the chipping, due to leakage from gates or stoplogs. After sandblasting, the concrete surface was washed with water and blown off with compressed air.

Laboratory tests were made of several proprietary bonding materials without indicating any advantage in this case over a well compacted mortar. In the absence of a distinct proven advantage, it was preferred not to introduce a plane of separation by foreign material. Available knowledge of bonding materials is greatly in need of research and our tests to date are not sufficient to rule them out. The following procedure was adopted:

A few inches ahead of the rising concrete level in the forms, the old surface was moistened and brushed with a thin coat of neat cement paste. A plaster coat of 1:3 mortar was then applied by hand by throwing it forcefully against the old concrete and following up by vigorous slapping with the palm of the hand, protected by rubber and canvas gloves. This technique was borrowed from L. W. Walter, Concrete Engineer of the Erie Railway. It is believed that the blow serves to cause relatively deep penetration of the neat cement into the pores of the old concrete. It produced a bond which, by test, was always stronger than the old concrete.

For reinforcing steel one-inch round deformed rods were placed on 12-in. centers both ways, 6 in. in from the surface and continuous along

the ogee slope. These were broken horizontally at construction joints. The location and spacing of construction joints furnished one of the major problems.

The new facing had a minimum thickness of two feet and average thickness of about 2 ft. 8 in. It is believed that with this thickness the frequent temperature changes which would alternately freeze and thaw the surface will not penetrate into the old concrete which should remain frozen practically all winter and, at most, experience only two or three cycles per year instead of many.

Repeated cycles of freezing and thawing in good concrete do not have any material effect except at the surface. If the underlying concrete is porous and weak, however, this protection should be of considerable value.

Experience to date indicates that the procedure as outlined has been successful and if, as anticipated, water pressure has developed in the old concrete against the new facing, no detrimental effect is evident.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.

Maintaining Concrete Structures*

BY FRANK W. CAPP†

INTRODUCTION

Concrete structures, like those built with other materials, are subject to the destructive attacks of weather and service. Like other structures too, they may require alterations to meet changing conditions. To make necessary alterations or extensions and to keep the structure in a proper state of repair is the field of activity of the maintenance engineer. If the American Concrete Institute, through such sessions as this, can help build up a technology in this branch of engineering, it will have added materially to its long list of important contributions to the field of concrete construction.

That general "good practice" in maintenance work has not been developed is evident by the numerous loosened patches, unsightly surface treatments, and crude attempts at repair with which the country is dotted. Some unsuccessful repairs may have been due to a lack of understanding of the original cause of the failure. Others are due to the inherent difficulties of the problem and still others to having been misled by claims in behalf of special methods or compounds of doubtful merit. Some of the special materials and methods offered have shown merit in meeting the requirements in particular structures. Such successes, however, have sometimes encouraged extravagant claims regarding them and resulted in their use in cases to which they were wholly unadapted.

DIAGNOSIS

A correct diagnosis of what is wrong is imperative for successful treatment. The location, nature, extent and probable future progress of the troublesome condition should be noted. The service characteristics of the structure, including the type, functions, exposure, loads and the like, should be understood.

A favorable time to examine a structure for defects is following a

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†Structural and Technical Bureau, Portland Cement Association.

rain while drains are still active but after some drying has taken place; still better, when a rain has been followed by a freeze or during a thaw.

An important first step is to determine whether the basic trouble is with the *concrete as a material* or with the *structural assembly* or a combination of the two. Failure to differentiate between these is apt to lead to the adoption of the wrong method of repair.

If the trouble is with the concrete as a material, it may be due either to mechanical forces or to chemical causes.

If the defect is structural, load conditions, foundation, temperature changes, drainage and related factors should all be studied.

Complex and numerous combinations of factors may exist, but so much of the poor performance of concrete structures has been due to (1) occluded water plus low temperature; (2) corrosion of reinforcing bars; and (3) unanticipated stresses; that these three conditions will be given principal consideration here.

Structures exist today in splendid condition after extended periods of severe service which demonstrate that older specifications, when carefully followed, produced concrete structures of excellent character. Under recent specifications which include up to date improvements in design and construction, most of the defects cited here will be inexcusable.

1. Occluded Water Plus Low Temperature

This combination is the most disruptive agency to which concrete is subjected and the one most common in climates having freezing temperatures. If proper resistance to this combination be developed, usually little or no fear need be felt concerning other forms of attack.

Fig. 1 which shows the "Relation Between the Freezing Point of Water and the Pressure Under Which the Water Exists" deserves attention. Normally, water freezes at a temperature of 32°F. and in changing to ice increases in volume about 9 per cent. Should this expansion be restrained by pressure after the water reaches a temperature of 32°F., it will remain liquid down to a temperature of —8°F. At 30°F. 2,000 p. s. i. must be applied to prevent formation of ice with its large expansion. At —8°F. the enormous pressure of 30,000 p. s. i. is necessary to prevent freezing.

Climatic exposure varies throughout the country as indicated in Fig. 2. Records of 200 U. S. Weather Bureau stations for the 5 years 1924-28 were plotted to give these curves of isoalternations of freezing and thawing per annum. The numbers of such alternations for various parts of the country are surprisingly high. Portions of structures in direct sunlight may have many more alternations than those indicated.

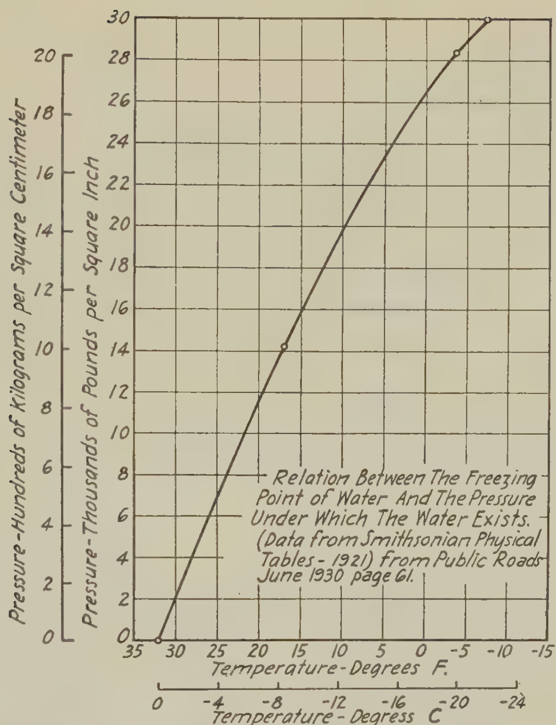

FIG. 1—THE RELATION BETWEEN THE FREEZING POINT OF WATER AND THE PRESSURE UNDER WHICH THE WATER EXISTS

The great number of such alternations in the intermountain sections of the West should be noted. Weather records from a few stations in Northern Canada show that the number of alternations there may be less than points farther south.

Severity of exposure on any portion is determined largely by the position it occupies in the structure. A few inches in location may make a lot of difference.

When water is entrapped within concrete in freezable concentrations and where there is no room locally for its expansion, freezing temperatures may do great damage. Such water may be held in large pores, in cavities, behind facings, under toppings and in joints. (Fig. 3.) In small cavities and pores of microscopic size water will not freeze except at temperatures lower than 32°F. However, when water freezes in the larger pores and spaces, the ice there forming draws water from the finer voids. Where a supply of water is available, as from wet earth, the water content of the portions of the concrete where freezing

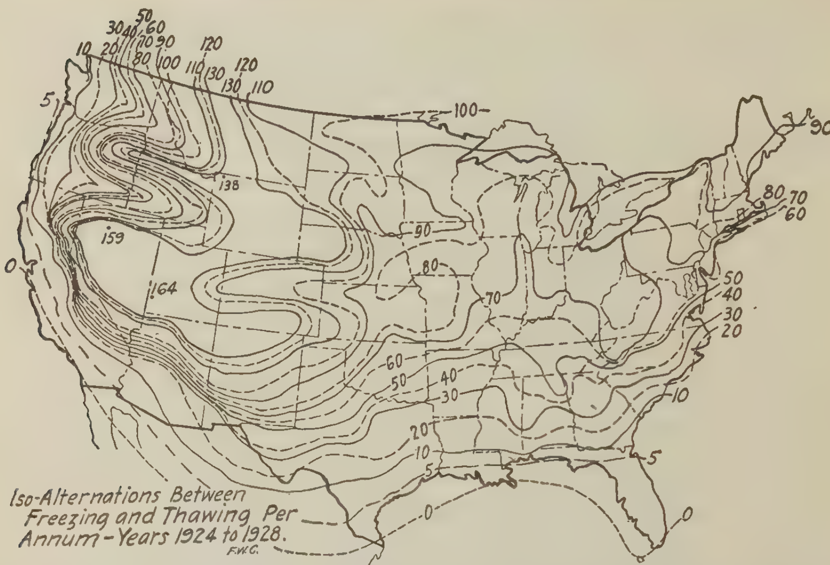

FIG. 2—ISO-ALTERNATIONS OF CLIMATIC FREEZING AND THAWING. BASED ON U. S. WEATHER BUREAU DATA FOR THE 5 YEARS 1924 TO 1928

is taking place is increased. A number of cycles of freezing and thawing may cause these portions to become so nearly saturated that the expansion of water while freezing cannot be accommodated and exerts pressures somewhat as indicated in Fig. 2. When such pressures occur locally, incipient breaks occur and tend to enlarge as ice crystals grow. Concrete of non-uniform texture is more susceptible to this kind of damage than that of uniform texture. Where breaks occur suddenly, the instantaneous release of pressure causes water to change to ice with explosive violence. Many phenomena observed in concrete so exposed, such as layering, the rounding of corners and edges, scaling and the like, may be readily accounted for in this way. An example of the great force exerted by the freezing of trapped water in large cavities is shown in Fig. 4. A 2-in. iron pipe 1 ft. from the face, set during construction, to observe temperatures during the setting of the concrete had not been plugged at the top and water had accumulated in its bottom. In freezing, this water had caused a flat, conical spall about 10 ft. in diameter.

Fig. 5 shows a spall 19 in. x 13½ in. x 6 in. deep which is typical of several along the same wall. In these cases water was entrapped in small metal boxes set to receive heads of anchor bolts. The concrete in both Figs. 4 and 5 seemed to be of good quality.

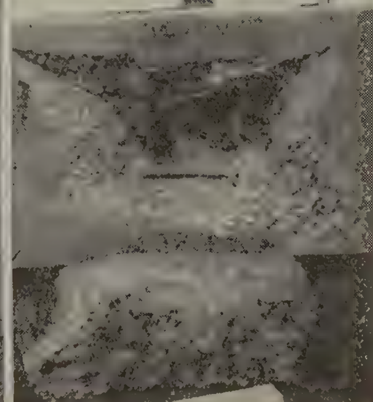

FIG. 3 (LEFT)—SEVERITY OF EXPOSURE VARIES WITH THE LOCAL SATURATION OPPORTUNITY. VARIATION IN THE DURABILITY AS AFFECTED BY PLACING METHODS SHOULD ALSO BE NOTED

FIG. 4 (TOP RIGHT) — THE TERRIFIC FORCE NECESSARY TO CAUSE THIS SPALL WHICH IS OVER 10 FEET ACROSS, WAS DUE TO THE FREEZING OF WATER IN 2-IN. PIPE 12 IN. FROM THE FACE

FIG. 5 (BOTTOM RIGHT)—METAL BOXES AROUND THESE ANCHOR BOLTS FOR THE RAIL COLLECTED WATER WHICH, ON FREEZING, CAUSED A NUMBER OF LARGE SPALLS ALONG THIS WALL SIMILAR TO THIS ONE

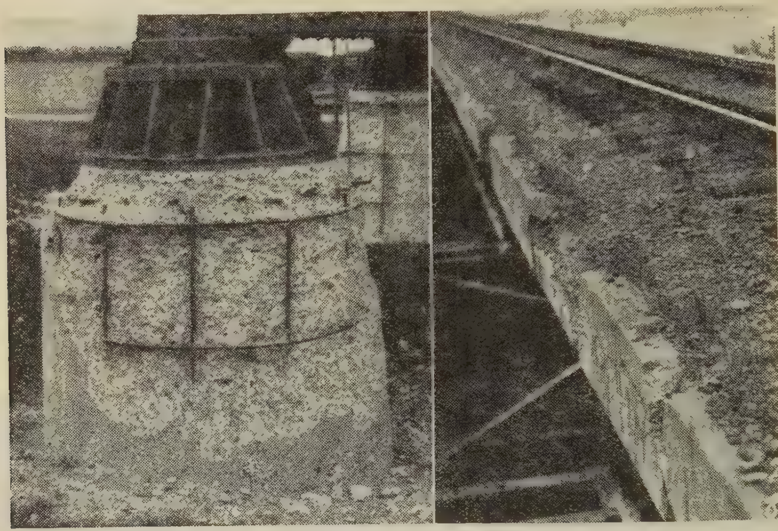

FIG. 6—THE DESTRUCTIVE EFFECT OF WATER FREEZING IN THE MINUTE CAVITIES OF A CHERT AGGREGATE CAUSED THIS DISINTEGRATION IN BOTH PEDESTALS AND DECK SLABS. THE DISINTEGRATED MATERIAL HAS BEEN REMOVED FROM THE PEDESTALS

An instance is shown in Fig. 6, for which the freezing of water in extremely small spaces was responsible, but here the cause is not so evident. These two views are from the same bridge. All the tower pedestals as shown on the left exhibit the same defects to a greater or lesser degree. The condition of the concrete in these is better near the ground, towards the interior, and on the sides down which water does not drain. The precast slabs in the right-hand view rest on the top flanges of steel deck girders and are so seriously disintegrated that passing trains frequently caused sizable chunks of concrete to fall. These slabs are about 9 in. thick and the gravel ballast they carry is commonly saturated with water. Their curbs and the bottoms outside the girders are exposed to severe temperature changes and wind. No evidence of disintegration was noticed during the annual inspections of this structure until 8 years after its completion. Since then, the disintegration has been at a progressively increasing rate.

The major cause of this disintegration has been traced to the use of a pit run brown chert gravel as aggregate. This, despite the fact that coarse pieces of gravel from the same source successfully withstood in the Laboratory 150 cycles of sodium sulfate treatment and the same number of cycles of freezing and thawing. Concrete test specimens

from this gravel withstood in the Laboratory less than 10 cycles of freezing and thawing, after which a visual examination showed the aggregate to be shattered and greatly at fault. Microscopic studies of the aggregate showed that it contained numerous relatively large cavities surrounded by material of fine texture. A number of the larger pieces of this gravel were submerged in a pan of water and a vacuum created during which a considerable amount of air was seen to leave the aggregate. Immediately following this, these same pieces were subjected to freezing and thawing with the result that they were badly shattered in from 5 to 10 cycles, about the same number as produced failure in the concrete test specimens made with this aggregate. This serves to illustrate the destructive effect of freezing temperatures when water is held in extremely small spaces.

2. Corrosion of Steel

The corrosion of reinforcing bars gives rise to some common problems in maintaining reinforced concrete structures. An appalling amount of carelessness in placing and protecting reinforcing bars is apparent to even the casual observer. (Fig. 7.)

Corrosion requires the presence of both moisture and air and is accelerated by chemicals such as the acids found in smoke, sea water, brine, etc. It is nearly as common in the south, particularly near salt water, as in the north. Consequently, the quality of the concrete specified for reinforced members for outdoor exposure should correspond to that further north.

Members of small dimension are most often affected. Bars in corners need more cover than those near flat surfaces. Bars of large size must be given more protection than small bars.

Alternating cycles of wetting and drying promote aggressive attack. Rust commences at some vulnerable spot where there is insufficient cover either due to thickness or to the poor quality of the concrete, and may progress to places otherwise properly protected. Main bars not properly supported during construction, tie bars and stirrups too close to the surface and corrodible chairs usually furnish the starting point for corrosion.

Where bars have sufficient strength of cover (due usually to depth) the pressure from the forming rust may be resisted, thereby excluding air and checking corrosion. For this reason, the prevalent notion that small shrinkage cracks are necessarily breeders of rust is ill-founded.

Fig. 8 shows a beam bottom in a railroad pier along the Atlantic seaboard. Of course, sea water was blamed for this. An examination, however, will show that the mortar from a harsh wet mix was strained

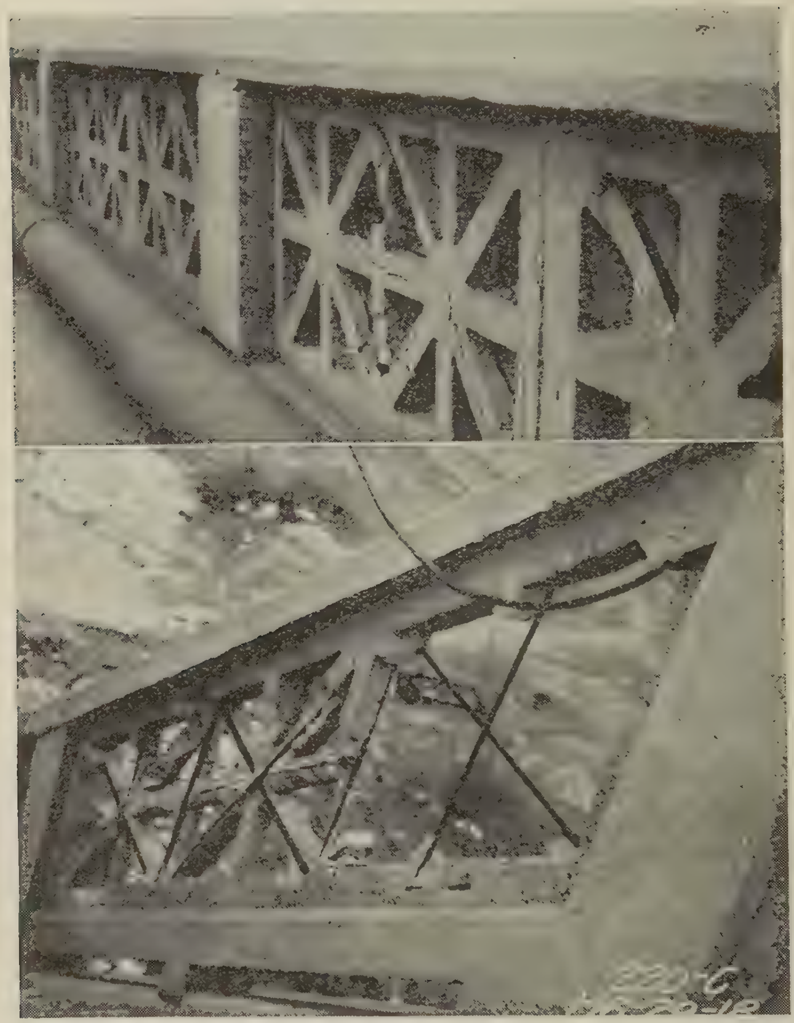

FIG. 7—THE CONCRETE WAS SPALLED BY THE CORROSION OF REINFORCING BARS ALONG TWO MILES OF THIS BRIDGE RAILING. WHERE PROPER PROTECTION WAS GIVEN, AS IN THE LARGE POSTS, THE CONCRETE WAS NOT AFFECTED

through the bars and coarse aggregate and was puddled so as to form a thin surface film. The concrete resulting afforded little protection against corroding influences. Such members are difficult to repair.

Fig. 9 is a close-up of cracks originating around a bar and illustrates the enormous force which may develop from corrosion.

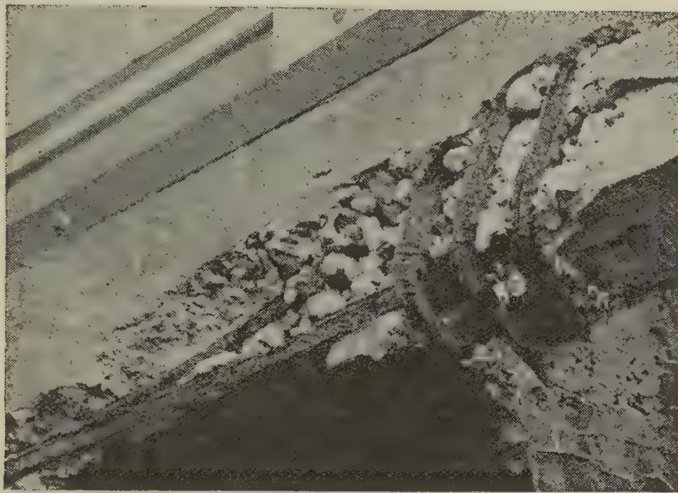

FIG. 8—RUST STARTED WHERE BARS HAD INSUFFICIENT COVER, AND PROGRESSED ALONG THE BARS. THE NON-UNIFORM CONCRETE RESULTING FROM PUDDLING HARSH WET MIXES IS OF INTEREST

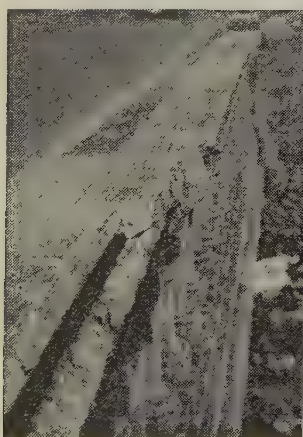

FIG. 9—CRACKS FOCUSING AT THIS BAR INDICATE THE FORCE PRODUCED AS RUST FORMS

FIG. 10 (RIGHT)—LEVEE PLACED AFTER THIS BRIDGE SUPPORT WAS BUILT CAUSED IT TO SETTLE AND CRACK THIS CONTINUOUS GIRDER

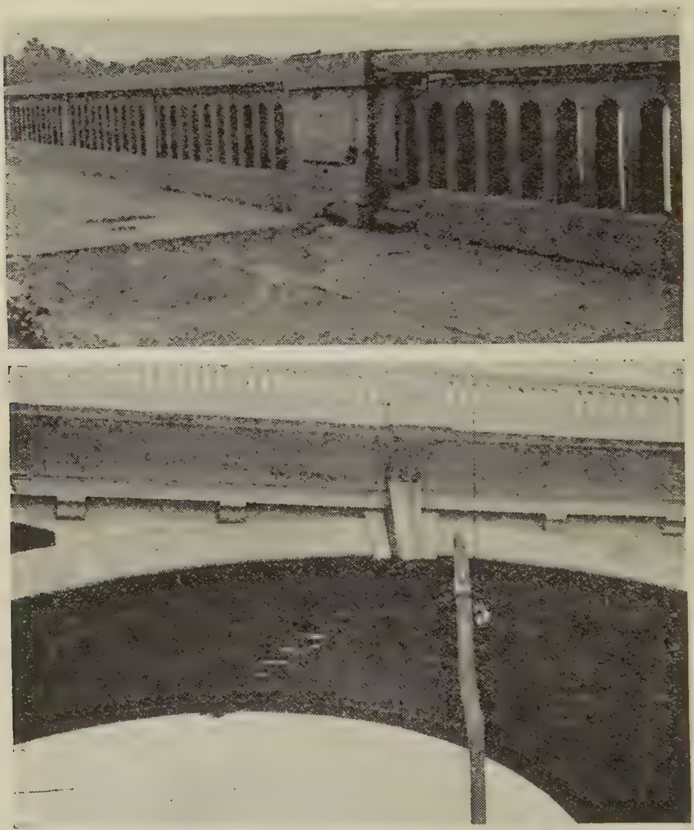

FIG. 11—THE OPENING PROVIDED IN THE DECK AND RAILING WAS LESS THAN BETWEEN THE ENDS OF THE CANTILEVER GIRDERS AND THIS DAMAGE RESULTED

3. Unanticipated Stresses

Some things that should not, but commonly do happen are:

(1) Foundations settle and cause terrific local stresses. (Fig. 10.)

(2) Hand rails, architectural features, pavements, walks and the like, sometimes are built monolithic with main members without having been designed to carry load. Deflection of the main members may force such minor and weak members into action and cause damage. (Fig. 11.)

(3) Splitting and tearing at joints due to expansion from temperature and the flow of plastic fillers is common. These fillers may not respond

to retreating expansion and permit infiltration of water and foreign material. Pressure through such soft and non-uniform material may produce numerous local shatter cracks. Pressure at such a joint (or crack) when water is present during freezing temperatures, is particularly damaging.

4. Accumulated deformations may be highly destructive and cause unsightly defects. A slippage under stress, a joint filling with solid material, any "dogging" preventing return, and soon something must give. Curved walls and so-called sliding joints are examples in which these conditions, often exist.

MAINTENANCE METHODS

Considerations in the repairing of structures are almost as varied as the conditions that have made the repair necessary. Most corrective measures are intended to insure the serviceability and extend the economic life of the structure. These may involve anything from complete restoration to original outline to makeshift repairs that meet the minimum requirements of safety and usability. The reason for repairing as well as the nature of the deterioration and its rate of progress must therefore be considered in each operation. Comparatively deep disintegration on a dam or massive structure in the wilderness may cause no concern, whereas even a light surface attack which disfigures the appearance may be highly objectionable in a structure in a city park.

Frequently, defects from poor workmanship which affect only the surfaces, corners, edges, fill planes and the like, are stopped by better concrete at shallow depths. Unless appearance is important, such defects can often well be left alone or action deferred indefinitely.

The maintenance methods to be followed in dealing with the three major causes of deterioration herein discussed are outlined below. In all cases the conditions responsible for or contributing to the deterioration should be corrected if possible. This is doubly important when the concrete is of inferior quality and incapable of offering proper resistance.

For Freezing of Occluded Water

The chief considerations here are:

(a) To correct those water conditions which keep the concrete saturated or permit water to be trapped or occluded;

(b) To discourage the non-uniform retention of water; and

(c) To reduce intermittent wetting and drying.

Drainage is often the most practical and effective method to accomplish all the above.

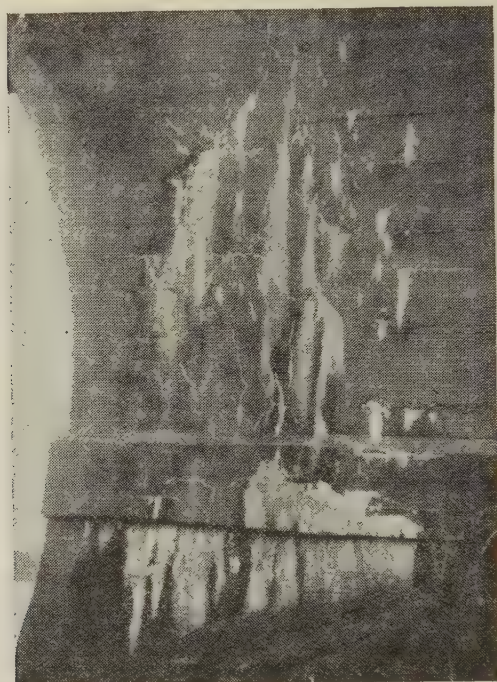

FIG. 12—WHERE REPAIRS ARE MADE WITHOUT ALSO CORRECTING THE DRAINAGE, SUCCESS CANNOT BE EXPECTED

A waterproofing membrane or other impervious surface treatments, applied so as to prevent the entrance of moisture, are advisable for some cases. Any surface treatment or method sealing water within the body of the concrete, or enclosing water by any portions thereof, must be carefully avoided in freezing temperatures. (Fig. 12.) When placed to prevent the entrance of water, punctures in the membrane and tears in flashings must be prevented.

"Keep the water out if possible, but if it gets in provide an easy way out"—is a good slogan for the concrete maintenance engineer.

Corrosion of Steel

The correction of water conditions here also is of prime importance particularly if wetting and drying are frequent. Little or no attention need be devoted to steel in members continuously saturated or below ground level, the latter only if certain electrolytic conditions exist.

Where bars have had insufficient cover and where clearances do not permit encroaching on the former outline, it will be necessary to cut completely around such bars and often replace them. If rust has progressed entirely around a bar so that it is bent outward, again it

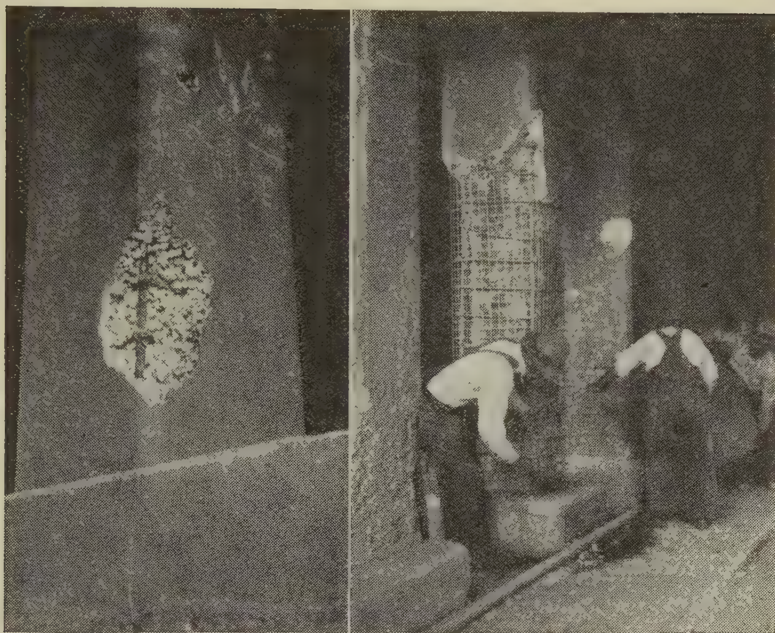

FIG. 13—COLUMNS OF "CRACKERJACK" CONCRETE AND REINFORCING BARS OUT OF POSITION FINALLY HAD TO BE REPAIRED

will be necessary to cut completely around the bar. New bars that are needed for strengthening and for replacing old ones must be set back from the surface far enough to insure proper protection and anchored and tied into the old concrete. (Fig. 12.)

Unanticipated Stresses

Here again the correction of the responsible conditions is imperative. Such usually will require a structural analysis.

Settling foundations must be underpinned or the bearing capacity of the ground improved. At times it may be possible to remove local damage caused thereby and repair as outlined for similar defects from other causes.

Where minor parts fail from carrying unanticipated stresses, they often may be relieved by cutting into sections and providing more clearance. Where plastic fillers or local adhesions have caused local defects, it is often feasible to remove them and provide spaces which are self-draining and self-cleaning.

Concrete for Repairs

Whenever new concrete is to be applied to old concrete the questions arise—"Will it stay put?" "What will it look like when it dries out?"

A good bond is essential to structural integrity. To achieve this, a cement film must attach itself to every particle of the receiving surface and be left undisturbed while adequate strength develops. Air or water films on any portion of this surface must not be permitted to form. To avoid this, a slightly absorptive condition of the receiving surface is commonly specified.

Another satisfactory method which has not received the attention it merits, is that of applying dry cement to the thoroughly wetted receiving surface. In this, dependence is placed on the pull between the water in the surface pores and the dry cement.

Patches should not be carried across active cracks or joints.

The volume change characteristics of the new concrete should correspond to those of the old in its present state so that differential movements will not take place and injure the bond. Because the receiving surface has undergone early adjustments, particular attention is necessary to make the new concrete of a low water-cement ratio, a low cement factor, and compacted more thoroughly than for new construction. For the sake of appearance the materials for the new concrete should be essentially the same as in the old unless the service shows these to be unsuitable.

The new concrete must be protected against displacement or slumping from inadequate support, over thick applications on steep surfaces, and from premature loading.

The new concrete must be protected and cured even more carefully than for new structures because the old concrete may absorb moisture too rapidly from the new material. Furthermore the temperature of the old concrete may be too low for the early development of strength. Curling of patches may easily follow and the bond be broken locally, particularly around its edges.

The technique for applying new concrete to old concrete and for making general repairs is well handled in the "Specifications for Repairing Deteriorating Concrete" 1933, of the American Railway Engineering Association. These specify the same materials as for other well controlled concrete, placed and protected by methods adapted to this type of work.

I shall therefore close by referring maintenance engineers to these specifications for details to be followed to satisfy the fundamentals herein discussed.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.

Studying the Durability of Concrete*

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INTRODUCTION

Back in dim pre-historic times when mankind first began to abandon the natural shelters afforded him by caves and hollow trees, he looked about for a cheap, easily handled, durable construction material from which to fashion his crude shelters. For centuries his wants were so simple that cairns of loose stone or mud-daubed thatch satisfied his needs. But with growing intelligence, and a mastery of the elements of fire and water, he sought more permanent building materials.

These wants are still unsatisfied, and today, with concrete acknowledged to be our most durable, reasonably priced material of construction, we find him dissatisfied with the product and seeking ways in which to make it even better adapted to resisting the elements. Could the engineer of less than a century ago have had assurance of a material even a fraction as economical to handle and as resistant to the elements as is our modern concrete, he would have thought that the millennium had arrived. For some reason our demands always keep ahead of our ability to supply them. Our paupers on relief revel in luxuries of which the early English kings never dreamed. We seek for social security, economic security, spiritual security, and durable construction materials, never quite catching up with our wants in any direction.

There is little doubt that concrete of today is of better quality than was ever before built, but the examination of our older structures which are beginning to show distress and deterioration indicates that the ravages of time and the elements will eventually destroy it, and we are still seeking means of improving its resistance to these forces, seeking some method of predicting what the next structure we erect will do as compared with those of our previous experience. In all walks of life we find that after men get old they look back upon the days of their youth and remember that it was attractive and rosy.

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They recall the spring of hope that forever flowed within their breasts, particularly in springtime, and are thoroughly convinced that the present generation is in a state of degeneration and the only possible hope is to return to the conditions as they were in their childhood. So in the field of construction, we find engineers old at least in mental processes if not in years, who are thoroughly convinced that our concrete of today is vastly inferior to the concrete of forty years ago; that we must return to the days of our youth and attempt to retrace our steps. Perhaps we *should* return to the days of our youth and retrace our steps. Almost any of us could in some ways, improve the resulting product, but this is impossible. We cannot return to the kind of concrete we built forty years ago even if it were desirable. Construction methods, labor costs, transportation methods and costs, the time element, all have so radically changed that we could not do it. The many excellent old structures which have been standing for decades indicate to some extent, what we may expect from high-grade concrete construction. We can still secure that type of construction and without question, many of our modern structures will give even better accounts of themselves.

EFFECT OF CEMENT ON DURABILITY

In studying the durability of our structures, and trying to devise means of producing more durable concrete, and to measure its probable durability, we will have to study both the past and the present, and try to anticipate the future. Let us begin with the present and start with the design of a concrete mixture which without doubt is one of the significant phases of the life history of a concrete structure. Concrete is composed of the cement, water, aggregate, time, temperature, and energy which went into the mixing and placing of the component parts. Probably the most important element in concrete is the cement, although as a matter of fact, it is almost as difficult to pick the most important component of a concrete mixture as it is to pick the most important component of our bodies. In predicting the probable effect of the cement on durability our studies quickly come up against a dense, blank wall of ignorance as to just what the properties of cement or what the composition of the cement may have to do with the probable durability of the concrete made from it. We do know this, however, that strong concrete is more durable than weak concrete; that concrete should not be easily permeated by water. That concrete cannot be much better than the aggregate of which it is chiefly composed.

Variations in the composition of the cement unquestionably affect to a certain extent, the strength of the resulting concrete; they affect

other properties of the concrete which in turn reflect upon other variables and probably do affect the durability and resistance to disintegration. It is now generally agreed that excessive quantities of tricalcium aluminate in a cement produce concrete having lower resistance to sulfate waters and sea water than concrete made from cement having smaller quantities of tricalcium aluminate. There are however, some very striking examples of concrete made from cement supposedly high in tricalcium aluminate which have given very satisfactory service when exposed to sulfate water. Recent studies reported by F. M. Lea indicate that perhaps we do not know what the compound composition of our cements really is unless we keep much closer control of our burning and cooling conditions than is customary. According to his studies, it is possible for a raw mix to be burned under such conditions that the actual percentage of tricalcium aluminate present may be as low as zero per cent, or as high as 15 per cent, depending upon the temperature of burning and rate of cooling and consequent glass formation. Further we do not know the significance of any of the minor chemical constituents always present in a cement mix. The geological formation from which mixes chemically identical may come might have some significance. Just now we find many engineers writing specifications limiting the constitutents of the cement with the thought of securing certain compound composition of the resulting product. Let us hope their results are satisfactory, but a careful study of existing structures and of the theoretical basis back of these changed compositions does not satisfy the author that we need expect any radical change in the quality of our concrete due to change in cement composition alone.

CEMENT IN COMBINATION WITH WATER

The cement in combination with water, serves two functions in a concrete mixture: It makes the mixture plastic or workable, if the gradation of the aggregate and the quantity of the paste are satisfactory, and after the mixture has been placed in position it sets or hardens and is responsible for the resulting strength of the finished concrete. Both of these functions are important, and have a bearing upon the phenomena of durability. There seems little question but that different cements have different plastic properties, depending somewhat upon the fineness of grinding, possibly their chemical composition, and upon other properties not yet known. Certain admixtures such as some of the waterproofing compounds which have been used in the cement or the paste (T. D. A., calcium chloride, and probably other materials) sometimes, when added in very small quantities, appear to have a marked influence upon the plasticity or workability of the

resultant mixture. Closely related to plasticity and workability is the property of cement which causes a concrete mixture to retain the water to a greater or less extent. Concrete mixtures which do not retain this water are said to bleed. Different cements have different characteristics as regards this action, and there is reason to think that this is intimately associated with the problem of durability of concrete and the resistance of the concrete to freezing and thawing and in some cases, to the action of sulfate water. In fact, it appears that many of the properties which enable the concrete to resist successfully the disintegrating effect of weathering, probably chiefly freezing and thawing action, also enable it to resist sea-water and alkaline waters effectively, but occasionally the reverse condition appears to be true. Some tests seem to indicate that concrete made with cements high in tricalcium aluminate, and presumably low in their resistance to sulfate action, are high in their resistance to freezing and thawing.

An examination of structures which have shown marked indication of disintegration due to weathering action, and of concrete which is failing under a freezing and thawing test, indicates that very frequently this failure consists of a loss of the bond between the mortar and the coarser aggregate. The sufficiency of this bond is apparently in some way closely associated with the property of cement which lends plasticity to the mixture, and which prevents bleeding. To what extent the compound composition of cement affects this phenomenon is unknown. The surface characteristics of the aggregate have a marked effect in this case, and there is fairly good evidence to indicate that some combinations of cement and aggregate can be improved by changing either the cement or the aggregate, or both. H. S. Mattimore's investigation (A. S. T. M., Part II, Vol. 35) indicates such a situation. There is also reason to believe that the ability of a concrete mixture to lose its water may be advantageous. Many examples of concrete construction, particularly architectural concrete, take advantage of this property. John J. Earley has reported to this Institute his method of casting concrete in absorbent molds which has without question produced concrete of unusual durability. Evidently the evil effects of bleeding which is a form of segregation or water loss, can be almost wholly, if not completely, off-set by the proper manipulation of the mass before final set occurs. The Johnson method of constructing concrete pavements makes use of this same principle. Any one who questions the durability of concrete made by this method need only go to Sioux City, Iowa where Mr. Johnson placed many miles of such concrete, to be convinced that durable concrete can be produced in this way. Unless some special precautions are taken, however, there

is every reason to believe that the accumulation of water around a particle of aggregate, particularly under the aggregate, has a very deleterious effect upon the durability of the concrete. Removing this water by means of pressure, by vacuum, by manipulation, or by absorptive forms or combinations of these methods, results in a definite improvement in the quality of the concrete.

There is reason to believe that the shrinkage of concrete is affected by the cement used. If the actual loss of water and consequent shrinkage which does occur should approach the ultimate which might occur, if completely dried out, variations in cement might produce concretes of significantly different qualities as regards shrinkage and durability. Engineers, however, have been prone to over-estimate the probable effect of this shrinkage by assuming that the full amount of shrinkage is likely to occur. Carlson points out (A. S. T. M. Proc. 1935, Vol. 35 Part II) that under ordinary conditions of exposure concrete sections of any appreciable size will lose their water and reach their ultimate possible shrinkage due to water loss so very slowly that in all except rather unusual conditions, this water loss will never occur. It is very likely that the variation in shrinkage due to differences in cement is nothing like as great as variations in shrinkage due to differences in aggregate, particularly as regards the size of the aggregate. Carlson also points out that the boundary condition between aggregate and paste appears to be an important factor in volume change due to shrinkage, which perhaps may account for a large portion of the differences in shrinkage that have been reported as due to different cements.

Another important element in concrete is water which, while a necessary ingredient in order to produce plasticity and provide for the chemical phenomena known as setting and hardening, is objectionable from every other point of view. There has been no information to indicate that any water recognized as potable water from point of view of its chemical content, is not satisfactory for concrete. In recent years, since the development of the water-cement ratio theory, there has been some tendency to use too little water, to attempt to place concrete mixtures which are harsh and really unworkable, and unless special means of handling and placing are developed, this lack of workability really produces a concrete less durable even though of high strength than with the use of more water. Where truly plastic workable concrete is being placed, durability follows closely the line of strength, and we need give but little further study to the problem of water in the original concrete mixture. We must however, carefully safeguard against segregation and bleeding, and if these do occur,

whether caused by either cement or aggregate, use means to handle the material so as to eliminate the dangers.

INFLUENCE OF AGGREGATE ON DURABILITY

Aggregate which constitutes the greatest part of the volume of our concrete, may be composed of fragments of stone from literally hundreds of different geological formations; the shape, surface characteristics, strength, and density of all these particles vary widely. Most concrete is placed with a natural sand which has usually been exposed to weathering for ages, and ones first thought would be that such material would, without question, be durable. Studies of the characteristics of aggregates, however, indicate that this may not of necessity be the case at all, and the quality of the sand or the fine aggregate will require very careful study before it can be given a clean slate and the user feel certain that concrete made with it will be durable. The coarse aggregate consisting of gravel or stone, is also an important element in concrete. In an average paving mix the absolute volume of the fine and coarse aggregate constitutes about 70 per cent of the total, and the quality of this aggregate plays a very important part in the durability of the finished product. Not only must the aggregate itself be durable and resistant to disintegrating weathering action, but it is the opinion of the author that the surface characteristics of the aggregate play an important part in the phenomena of bleeding, and the development of an unsatisfactory contact between the mortar or paste, and the aggregate particles. No tests have yet been developed to show this property.

We have developed reasonably satisfactory tests for the durability of aggregates by means of sodium sulfate or magnesium sulfate tests, or by actual freezing and thawing tests. These tests definitely pick out and indicate as unsatisfactory, certain aggregates that have in the past caused unsatisfactory concrete structures. Without doubt there are borderline cases where such tests are unreliable and for some time the application of such tests to the selection of aggregate must be given careful study before the material is either accepted or rejected, and in every case, if possible, the behavior of existing structures using this aggregate should be studied. In studying such structures one must be careful to recognize the degree of exposure to which a structure has been subjected before forming an opinion as to the original character of the concrete and the aggregate from which the structure was built.

The elastic properties of aggregates vary widely, and the elastic properties of the resulting concrete will follow rather closely the elastic properties of the aggregate of which it is composed. The thermal

coefficient of expansion of aggregates will vary widely and again the chief variable in a concrete as to its thermal deformations is found in the aggregate rather than in the cement. The engineer is usually in ignorance of what particular aggregate will be used in construction and he designs for average conditions. But it must always be borne in mind that these properties may be very important factors in determining the satisfactory life of the concrete structure. This is particularly true if the structure, like a pavement, requires the building of expansion joints to relieve such deformations. If adequate provision is not made the high stresses develop a type of failure often mistaken for material disintegration.

TIME, TEMPERATURE AND WATER

Time, temperature and water are elements which enter into the design of a concrete mixture. It is difficult to separate the three, but without doubt, the proper relations of time and temperature with moisture play an important part in developing a high-grade concrete. If satisfactory temperatures for the proper hydration of cement are not maintained for a sufficient time to permit proper hardening, it is almost certain that the full durability and strength of the concrete will never be developed. The writer has seen many structures in which blame for unsatisfactory results was placed upon cement or aggregate or other variables when as a matter of fact, the trouble was due to unsatisfactory temperatures and time of curing. These things are so well known that it seems useless to mention them, but you will be surprised at the calibre of engineers who still neglect such factors.

PLACING METHODS

The other element mentioned as entering into the design and placing of the mixture, and which too many of us forget, is the energy used in placing and handling. Some of our older concrete structures which are making such an excellent showing as to durability even under adverse conditions, were placed with the old-fashioned methods of ramming and tamping concrete into place. The mixture never was plastic or workable, but in the type of construction in which it was used, reinforcing, forms, and other details were such that it was possible to force the concrete into position and compact it properly. The result was a low water-cement ratio, a minimum of voids, and a high degree of impermeability. The old timer who wants to go back to the good old days of low strength, non-uniform, coarse ground cements sometimes forgets this, one of the most important elements in that type of structure. We did not have any really bad examples of lack of durability of concrete until we developed machine mixing, and troughs for the conveyance of concrete. The speeding up

of the concrete operations, mechanical mixing spouting and flowing into place from a common point brought a whole train of evils in their wake, all of which are familiar to every engineer and builder, but from which we have not yet fully recovered. Much as the cement and the cement industry have changed, the changes in our methods of mixing, handling, placing, and finishing the concrete have been even greater, and all of these are elements which enter into our resultant concrete structure.

VARYING EXPOSURES

Another thing which we forget is the extreme variation in the degree of exposure to which different structures may be exposed, and as we go back and look at our earliest examples of concrete construction, it is just possible that some of those placed under extremely adverse conditions have been lost sight of and forgotten, and that some of the excellent surviving examples, never did have severe exposure. I recall recently reading an article by a journalist about the engineering ability of the primitive Pueblo and Cliff-dwelling Indians of the southwest. He cited some of their structures built with sun-baked dobe still in really excellent condition, bearing the finger prints of the little Indian children who patted the moist clay into place, and then compared such construction to some of the irrigation structures which the U. S. Reclamation Service had built in that vicinity only a few years ago, some of which were showing disintegration and distress. He did not mention that these primitive structures built perhaps 800 years ago, are there only because of the almost perfect protection they have received from the overhanging cliffs beneath which they are nestling. We engineers are not that bad, but some of us approach it in our ridiculous attempt to interpret the results of concrete disintegration. Perhaps we have a dam, or retaining wall, or an abutment—to take a specific case, the author will use his own experience examining a structure on a railway system where there was available unusually good engineering data as to materials used, methods of construction, etc. We examined a 30-foot concrete arch which had been placed about 30 years ago. Considerable disintegration showed around the springing line of this arch and the junction of the wing-walls and spandrel-walls where water had evidently been accumulating and slowly seeping through, with the resulting disintegration of the concrete mass. We examined this structure with a great deal of care; picked off portions of unsound concrete, studied the stone and sand particles involved and made note of the brand of the cement used, who the contractor was, and tried to arrive at the cause of the failure. A mile away was a similar arch across a similar stream, which appeared to be in perfect

condition. Had we been content to stop with the superficial appearance of those structures, we would have been content at once to say that one concrete was much more durable than the other. A study of the records indicated that the two structures were built simultaneously by the same contractor, using cement, sand and stone from exactly the same sources. He used an abundance of water in both. The difference in their appearance after thirty years of exposure was entirely due to the difference in exposure to which they were subjected. The first structure was under a comparatively shallow fill at a sag in the grade line. The ballast pocket which develops under almost any roadbed after years of service, was serving as a trough to carry water for half a mile or more down to the arch. There it soaked through the more or less porous back-fill which covered the arch and lay back of the wing-walls and kept the concrete mass thoroughly saturated throughout the years. The mixture was lean and the water was slowly seeping through, with the result that freezing and thawing and possibly crystal from salts in solution in the ground water, was gradually breaking down both the mortar and the stone. The other structure was under a much deeper fill, was at a high point in the grade instead of at a sag, and there was no water back of it at all. Conditions of exposure can vary very widely even in the same structure. A pavement slab going up a long, flat grade, of identical concrete throughout may show marked distress at two or three points where it passes over a ledge of rock which is draining into the foundation of the roadbed, producing frost-heaving in freezing weather and maintaining always a thoroughly saturated condition in the concrete during winter weather. I have read on engineer's report upon the lack of durability of certain retaining walls where disintegration was observed around an expansion joint which was leaking badly, the concrete breaking up and falling off for several feet on each side of the joint, but was perfectly sound and durable a few feet farther out on either side away from the failed portion. He tried to explain this failure by pointing out that unquestionably the cement varied in quality. In this case there were at least two different kinds of concrete in that structure. It is almost certain that the builder in attempting to get a satisfactory appearance around his expansion joint, either changed the mix and provided for wet, more fluid concrete at this point, or else took advantage of the natural segregation that probably occurred and puddled the slop over to this end. An examination of the concrete which was failing and comparing it with that which was still in sound condition would show that the mortar content was much higher near the joint and that without doubt the cement had nothing

whatever to do with the failure. The *Engineering News Record* reported only a few years ago, a bridge failure in Germany in which after a very careful study of the nature of the water in the river in which the pier had collapsed, and a very careful study of the cement used in this structure and its resistance to solution in such water, it was found that the failure had really been caused by improperly placing the concrete under water with the result of the formation of about three feet of laitance in the body of the pier. These examples are extreme illustrations of the failure to appreciate the possible variations in the degree of exposure and the uniformity of the concrete which we may be comparing. Not many engineers are guilty of such oversight, but in the finer gradations or variations in kind of concrete and degree of exposure, it is difficult to draw the line. For example, most engineers, including the author, believe that there is sufficient water rise in a 6 by 12 inch concrete cylinder so that on a freezing and thawing test we will expect the upper end to show less resistance to disintegration than the lower end. At the same time, we realize that when we finally finish off the upper end of the cylinder, we deliberately increase the paste and water content in order to secure a proper finish on the end. The author has tried many times to catch a difference in the top and bottom of pavement slabs actually placed in construction, and has as yet been unable to notice any apparent indication of water rise where the pavement has been properly finished. On some pavements which have shown definite disintegration this evidence of water rise has been marked, both in the structure and in tested cores.

We can learn a great deal about concrete and its durability by carefully studying our old structures; how they were built, and from what they were made, and under what conditions they have been exposed during the years that have passed, but we can also draw very erroneous conclusions unless we analyze data with the utmost care.

PREDICTING DURABILITY

The magnitude of the task ahead of an engineer in trying to predict the durability of his structures to be built from cements which differ considerably from the old cements, with new sources of aggregate which have been developed due to the increased demand for such material, and to be built by different construction methods than were used in the structures which he examined, has led him to try to develop some satisfactory method of predicting the probable durability of a concrete mixture under various conditions of exposure to freezing and thawing, temperature changes, alkaline and sea water. Within the last ten or fifteen years many tests have been proposed along these

lines with the result that some are now trying to standardize these tests. Useful as the tests have been in helping us to appreciate the significance of certain variables in our materials, and our concrete, upon its probable durability, we can readily commit some serious errors by attempting to standardize, or to lay down arbitrary rules at this time. The writer seriously questions if we have yet made a sufficient study of the case so that we can say just how many cycles of the magnesium sulfate test, or the sodium sulfate test will be required to assure the user that the aggregate should or should not be used. The same may be said of artificial freezing and thawing tests which have recently been coming into considerable prominence, and which the writer has been studying intensively for some ten years. The author particularly objects to trying to express the results of some of these tests quantitatively. He appreciates fully, having been in the situation himself, how difficult it is to interpret on a commercial routine basis the results of a soundness test and how important it is to be able to say to a producer, "your stone failed by such and such a per cent." It is very difficult to set such a numerical value for the type of phenomena that actually occur. When coarse aggregate fails under freezing and thawing tests, the action usually takes one of three forms:

(a) The stone or gravel breaks into two or more fragments with terrific force sufficient to disrupt large masses of concrete and disrupt the whole mass if many such particles are present. Unsound chert produces such results. This form of failure is serious, but does not lend itself to easy grading of the results.

(b) The mass softens and looses its strength with practically no volume change. Such a failure does not damage the adjacent concrete mass unless such pieces form a considerable percentage of the total. Such loss can easily be expressed quantitatively.

(c) Another type of failure consists of the gradual loss of particles from the surface of the aggregate; such loss occurs with little volume change, but breaks the bond between the mortar or paste and the aggregate.

To try to express losses of these types with a common index value is impossible. So many variables enter into the results which will be secured by any of these tests that really to place confidence in the final results secured, enough specimens should be examined so that the results may be analyzed statistically. The writer is continually bombarded with the question "how many cycles of freezing and thawing will represent a useful life for twenty years," or some other period. There is, of course, no answer to such a question. One would have to

define not only the freezng and thawing test very carefully, but even more carefully, the degree of exposure to which the structure will be submitted. Let us examine together, some of the variables that enter into a freezing and thawing test and see why it is that for borderline cases it is very difficult to reproduce results in one laboratory, let alone check results in different laboratories. When a concrete test specimen is placed in a chamber to be frozen, and later removed to be thawed out, we have variables immediately confronting one as to the type of specimen to be used, and the procedure to be followed. First, at what temperature shall it be frozen? The immediate answer might be, at the temperature at which it will be frozen under natural conditions. But under natural conditions it takes a good many years to arrive at a conclusion as to whether a concrete is or is not resistant to weathering as represented by freezing and thawing. So we must at once decide to speed up the freezing and thawing test by using both lower temperatures during freezing, and also more intimate exposure to the cold medium to allow for a satisfactory rate of heat exchange. The size of specimen will become important here; the larger the concrete specimen, the greater will be the differential of temperature between the inside and outside of a specimen under a given exposure to low temperature, and accordingly, we will have set up greater internal stresses on account of this difference even before freezing and thawing has started. The outside portion will be frozen for a considerable length of time before the inside portion is frozen, and stresses will be set up under these conditions which will vary considerably from what would happen in nature where usually saturated concrete subjected to freezing and thawing is exposed on one face only. Heat flows out of concrete under such conditions for an appreciable length of time before even the outside portion is frozen, and when freezing on the outside actually begins, there is probably nothing like as great a temperature differential as one would find in a large test specimen frozen at a rapid rate in the laboratory. If a smaller specimen is chosen, the temperature differential from inside to outside is much lower, there is probably less stress, and the concrete will freeze quickly. It would seem that the small specimen might have some advantage over the large specimen, but the presence of unsound aggregate in a small specimen, or an excess of water at the boundry lines between the paste and the aggregate, producing weakness, will more seriously damage the small specimen. Such a place of weakness might cause early failure of a specimen, whereas a similar place of weakness in the large specimen might have practically no effect whatever. That the rate at which freezing and thawing, particularly freezing is done, is

an important element in freezing and thawing tests, is unquestioned. In some recent cooperative tests conducted by the Highway Research Board, laboratories conducting the freezing and thawing test at temperatures only slightly below freezing, and requiring 24-hours for a cycle, report but very little effect after 200 or more cycles. In other laboratories where more accelerated rates of freezing have been carried on, complete failure of some specimens has been effected in less than 200 cycles. With such significant differences in the results secured, it can be seen that freezing and thawing tests must be interpreted with care. The rate of freezing and thawing is dependent not alone upon the temperature in which the specimen is placed, but upon the way in which it is exposed to the temperature and the rate at which the heat may flow from the test specimen into the refrigerant. The size of the chamber in which a specimen is placed, the relation of the surface area of the specimens to the total area of the chamber walls, and the medium which surrounds the specimens, and forms the contact with the refrigeration coils in the chamber, are all important factors in determining how rapidly freezing will occur. An excellent analysis of the general problem by Hans Kostran appears in *Bauwirtschaft*, Vol. 1, No. 5, July 15, 1933, in an article entitled "Testing the Resistance to Freezing and Thawing of Building Materials." Dr. Kostran summarizes the influences which may affect the results in a chart which not only shows the variables, but their inter-relations and evaluations of their relative importance. (Fig. 1.)

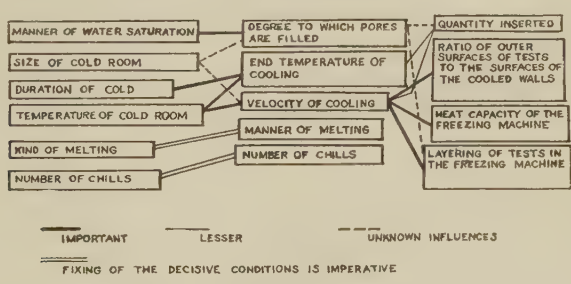

FIG. 1

Particular attention should be paid to the problem of water saturation and velocity of cooling. The velocity of cooling not only has a marked effect upon the time required to conduct a test, but also upon the severity of the test. Water saturation involves the manner in which the water is allowed to enter the concrete, the degree to which the pores are filled with water, and the actual quantity of water which may be inserted in the specimen. Not only is this an important factor in the experimental investigation of this character, but it is also im-

portant as effecting the actual resistance of the structure when in service. Some types of exposure are such as actually to force the water in under pressure, while other types are such that almost the only water present in the concrete will be that brought into the mass by capillary action or that which has been retained in the concrete since it was originally produced. We have found in our laboratory that if specimens are saturated with water by means of alternate pressure and vacuum methods, considerably more water is inserted into the specimen and the severity of the action increased.

The rate of cooling is influenced by the temperature of the refrigerating room, the ratio of the outer surface of the test specimen to the surface of the chamber through which heat passes from the specimen into the refrigerant, the medium through which this heat travels in going from the specimen into the refrigerant, and the capacity of the refrigerator. Low initial temperature in the refrigerator will not necessarily insure a rapid rate of freezing unless some means is provided through which the heat can flow from the specimen into the refrigerator. If this flow of heat has to occur through a comparatively large dead air space, the rate may be much slower than would occur where temperatures were considerably higher, but with the specimen placed close to the walls of the refrigerator and a better heat conductor, such as alcohol or brine solution, used as a medium through which the heat could flow. The temperature to which the specimen is lowered should be sufficient to insure the freezing of the water contained in the pores which probably act to a greater or less extent, similar to closed vessels, and as the pressure rises due to the formation of ice, the pressure might become sufficient to stop the formation of ice unless the temperature were lowered below the freezing point. At 2200 atmospheres the freezing point of water is —22° C. If the pores are not fully filled and freezing occurs rather slowly, it is perfectly feasible that as the pressure rises due to the formation of ice, the water flows over into these still available air spaces and only very low internal pressure is developed. The expansion of ice is approximately 10 per cent, so if the pores are only about 90 per cent full of water, it would be possible to freeze the water with very little internal pressure. Some porous, permeable concrete as well as some porous absorptive permeable building stones, brick and tile have very good weathering characteristics. Dr. Kostran points out that this may be explained by the above facts. Granting that no internal pressure may have been developed because of the insufficiency of the filling of the pores or their size and shape, there will still be stresses produced in freezing or thawing a concrete containing aggregate having a variable coefficient of expansion from that of the

mortar and paste. If these stresses are an important factor in the failure under the freezing and thawing test, it is obvious that the more rapid these changes occur, the more severe will be the action. It was with this thought in mind that the research reported by Professor Dawley* was initiated at Kansas State College. Whether Professor Dawley has found the thing which we started out to find may perhaps be another question. So often in research, we try to answer one question and in doing so, raise a dozen more. Time will not permit a complete discussion of all the variable items in any durability test, or any normal exposure test. Careful consideration of some of the points which have been raised will, in the author's judgment, prove helpful and instructive to any one designing concrete, maintaining concrete structures, or attempting to predict the probable durability of the material which he is using.

The apparent lack of decision and of positive information that permeates this paper (and which should be included in any paper which discusses the durability of concrete) is not peculiar to our field alone. For generations, employers have been endeavoring to secure good men, men of honesty, integrity, loyalty, and character, capable of handling their own particular work which they are doing, and making satisfactory contacts with other men with whom they must associate. Any survey of the factors which lead to success in any of the walks of life will enumerate the above as essential characteristics, but we don't know how to measure them nor how to develop them in our children, our students, nor our employees. Durable concrete is concrete of character, to secure it we must have faith and hope, and continually strive from every point of view to overcome its defects; being, as with our relations to man, always critical, always considerate, and always helpful.

*See page 609.

For such discussion of this paper as may develop readers are referred to "Supplement," Journal *for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.*

Alternate Heating and Cooling of Mortar*

BY E. R. DAWLEY†

INTRODUCTION

The Applied Mechanics Department[1] has been investigating the durability of concrete for the Engineering Experiment Station of the Kansas State College for more than 15 years. The tests, which included freezing and thawing, permeability, volume change, field specimens in alkali water, and finally heating and cooling have been made in an effort to explain why some concrete is more durable than other concrete. The Department of Applied Mechanics has been a pioneer in the investigation of the effect of the freezing and thawing of concrete. Always in the development of the work the question has arisen as to how much of the destructive action of freezing was due to the temperature change, and how much was due to the expansive force of the ice forming within the specimen. Heating and cooling afforded a method of divorcing the effect of temperature change from the effect of ice formation. In an attempt to answer this question several years ago the author took the first piece of concrete at hand which was a replacement plug used in core drilling, placed it in a tank of water, and ran live steam into the water for 15 or 20 minutes, until the temperature became constant; then ran tap water into it until the temperature again became constant. After repeating this process six more times, the specimen was inspected and the surface was found to be badly cracked. The present series of tests was the outgrowth of this original experiment; the object being first, to discover if the specimens after heating and cooling were really as adversely affected as far as strength was concerned, as their surface appearance indicated; and secondly, to discover if possible, the effect of using different brands of cement, different aggregates, and different cement contents.

Preliminary heating and cooling tests developed the following information:

1. The cracking of the surface and the formation of a ridge of calcium carbonate approximately following the crack is a universal characteristic of this kind of test.

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936.
†Professor of Engineering Materials, Kansas State College, Manhattan, Kan.

2. Concrete made with crushed rock as coarse aggregate was not as badly cracked as mortar.

It was decided to use mortar specimens, since speedy results were desired. A temperature change of 80 to 100 degrees was adopted as a compromise between simulating natural conditions and obtaining fairly rapid disintegration. The end point of the test was arbitrarily set at 4000 cycles.

MATERIALS AND APPARATUS

All specimens were 3 by 6-in. cylinders. Seven cements were used: Cements 1, 2, 4 and 5 were early strength cements, Cement 7 was a special cement, and Cements 3 and 6 were ordinary portland cements. Tables 1 and 2 show the physical and chemical characteristics of these cements. The chemical analyses are those of the manufacturers. Table 3 shows the computed compound compositions. The aggregate

TABLE 1—RESULTS OF PHYSICAL TESTS ON CEMENTS

Cement Number	Fineness, Per Cent Retained On No. 200	Soundness	Time of Setting				Tensile Strength, lb. per sq. in.		Surface Area, sq. cm/gm.
			Initial		Final				
			Hours	Min.	Hours	Min.	7 Days	28 Days	
1	19.28	O. K.	6	00	7	10	377	407	1660
2	.46	O. K.	2	00	3	15	275	370	2200
3	4.68	O. K.	3	30	5	15	343	460	1760
4	1.80	O. K.	1	45	2	45	357	440	2435
5	1.56	O. K.	2	00	3	10	280	388	2480
6	10.10	O. K.	3	00	4	50	347	465	1750
7	1.88	O. K.	2	45	6	00	395	450	1720

TABLE 2—CHEMICAL ANALYSIS OF CEMENTS

Cement	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Loss	Total
1	6.88	41.01	14.43	36.44	1.30	0.20		100.26
2	19.82	5.36	2.92	65.40	1.70	2.08	2.90	101.67
3	20.20	6.41	3.21	64.52	2.50	1.70	1.57	100.11
4	19.68	5.52	2.82	64.90	3.34	2.40	0.60	99.26
5	19.72	6.14	1.96	67.52	1.24	1.89	1.51	99.98
6	21.64	5.82	2.70	65.24	1.03	1.70	1.52	99.65
7	20.80	6.13	3.07	65.80	0.76	1.88	1.20	99.64

TABLE 3—COMPUTED COMPOUND COMPOSITION BASED ON TABLE 2

Cement	Per Cent $CaSO_4$	Per Cent C_4AF	Per Cent C_3A	Per Cent C_3S	Per Cent C_2S
2	4	9	9	69	5
3	3	10	12	57	15
4	4	9	10	67	6
5	3	6	13	68*	5
6	3	8	11	53	22
7	3	9	11	59	15

*Free lime estimated 1.75%.

was Kansas River sand, a clean sand of granitic origin weighing 111 lb. per cu. ft., and having a specific gravity of 2.61.

The gradation was as follows:

Sieve	*Per Cent Retained*
3/8	0
No. 4	0
No. 8	18
No. 16	52
No. 30	79
No. 50	95
No. 100	99

Gradation Factor 3.43.

All proportions were by volume and were 1:4.5, 1:3.5, and 1:2.

The water used in mixing the specimens and in which the specimens were immersed during heating and cooling was college tap water. The college water supply with a hardness of about 435 parts per million, is augmented by a varying percentage of treated city water with a hardness of 120 parts per million. A typical analysis of the mixture would show—

Total hardness	393 parts per million
Calcium	290
Magnesium	103
Alkalinity	350
Chlorides	19
Sulphates	74
Iron	0.8

The apparatus used for heating and cooling consisted of a copper tank 12 ft. long, 39 in. wide, and 10 in. deep, divided into three separate parts, each with separate steam and water inlets and drain (Fig. 1). The drains were at a height of seven inches giving a depth of water of one inch above the top of a 3 by 6-in. cylinder standing on end.

The steam and cold water are alternately admitted through the pipes in the right rear corner of each compartment. The baffles direct the flow so as to give a low temperature gradient past the specimens. The discharge is at the right front corner of each compartment. Solenoid valves alternately admit steam and cold water to the tanks (upper left hand corner, Fig. 1). These valves are controlled by an electric clock movement which operates a mercury switch by a cam action. The interval is 30 minutes for steam, then 30 minutes for water.

TEST METHODS

Thirty-three 3 by 6-in. cylinders were molded, eleven to the batch, for each of the seven cements in each of the three mixes: 30 of these were for compression tests, three were fitted with a stainless steel plug in each end for measuring change in length. Ten compression cylinders and one volume change cylinder were made from each batch.

All specimens were left in the molds 24 hours and then stored in the moist room for 27 days. Twenty-eight days after molding, three cylinders (one from each batch) were tested in compression. The

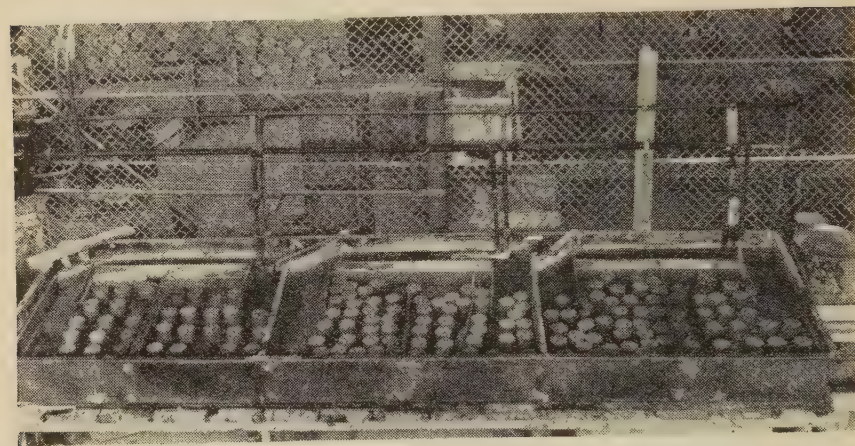

FIG. 1—HEATING AND COOLING TANK IN WHICH 3 BY 6-IN. CONCRETE CYLINDERS ARE ALTERNATELY COOLED TO 70° F., AND HEATED TO 166° F.

lengths of the specimens with plugs were measured at 68°F. and they, with the remaining compressive cylinders (30 in all), were placed in the heating and cooling tank on the cold cycle. As heating and cooling progressed the specimens were moved from left to right across the compartment, and turned over endwise to compensate for any temperature gradient either horizontal or vertical in the tank. Temperatures fluctuated with the season, and with failure of steam or water supply. The average high temperature was 166°F., and the average low temperature was 70°F., giving an average change of 96°F. per cycle. A duplicate set of specimens was made for part of the series and heated and cooled in a separate tank at a temperature change of 50°. This was done to see whether the damage was done by the high temperature side of the cycle, since 166°F. is hotter than most concrete ever becomes in service.

Specimens with end plugs were measured for change in length every day at first, and every two weeks after the change became smaller. These measurements were always made after the specimens had been brought to a temperature of 68°F.

The effect of the lime content of the college water on the formation of free lime was investigated* and it was found that free lime formed at about the same rate in specimens stored in distilled water as in those stored in college water.

*Unpublished Masters Thesis of P. F. Warner, 1935, Kansas State College Library.

TABLE 4—DATA OF TESTS

Cement No.	W/c Ratio by Volume	Wt./cu. ft. lb.	Cement Factor bbl./cu. yd.	28-Day Compr. Str. lb./sq. in.	Slump Inches	Average Per Cent Increase in Length, Cycles					Per Cent Of 28-Day Remaining After, Cycles		Per Cent Gain in Weight After 4000 Cycles
						200	1000	2000	3000	4000	200	4000	
						Mix 1:4.50 By Volume							
1	1.17	139.0	1.41	2480	2.25	0.080	0.131	0.150	0.195	0.237	29.1	26.9	3.10
2	0.96	134.3	1.38	3160	2.00	0.195	0.671	1.326	1.607	1.777	51.2	25.6	5.78
3	1.10	137.5	1.40	2030	2.00	0.056	0.143	0.375	0.478	0.592	89.3	92.4	4.84
4	1.12	137.5	1.40	3060	2.25	0.070	0.845	1.616	1.898	2.023	69.8	16.5	7.20
5	1.09	137.0	1.39	3240	2.00	0.056	0.148	0.240	0.381	0.556	75.1	38.9	5.32
6	1.18	136.6	1.38	2160	2.50	0.056	0.115	0.230	0.353	0.482	90.8	73.3	6.33
7	1.05	135.9	1.39	3080	2.00	0.056	0.091	0.150	0.208	0.265	72.5	64.8	5.49
Av.	1.10	136.8	1.39	2744	2.14	0.081	0.306	0.584	0.731	0.847	68.3	48.3	5.44
						Mix 1:3.5 By Volume							
1	0.87	140.2	1.76	4710	2.25	0.076	0.209	0.232	0.292	0.374	36.1	22.1	3.90
2	0.79	135.8	1.72	3770	2.00	0.133	0.718	0.982	1.155	1.262	70.5	34.6	4.60
3	0.87	139.4	1.75	3100	2.12	0.061	0.127	0.201	0.263	0.314	90.9	108.5	5.07
4	0.82	138.0	1.74	5460	2.12	0.092	0.378	0.649	0.937	1.243	63.0	24.6	5.57
5	0.84	138.1	1.74	4760	2.50	0.064	0.142	0.205	0.285	0.374	74.1	42.8	4.23
6	0.87	138.0	1.73	3420	2.16	0.056	0.095	0.175	0.263	0.371	88.7	70.1	5.02
7	0.81	137.2	1.74	4340	2.00	0.066	0.109	0.142	0.203	0.232	78.0	80.1	4.70
Av.	0.84	138.1	1.74	4223	2.16	0.078	0.254	0.369	0.485	0.596	71.6	54.7	4.73
						Mix 1:2 By Volume							
1	0.49	144.8	2.82	6840	2.50	0.103	0.192	0.280	0.382	0.453	63.4	48.7	5.22
2	0.56	141.4	2.72	5630	2.11	0.092	0.353	0.460	0.594	0.743	90.5	71.9	4.53
3	0.56	142.5	2.75	5650	2.25	0.045	0.075	0.088	0.120	0.124	93.1	139.4	3.83
4	0.54	142.5	2.75	7030	2.25	0.059	0.128	0.195	0.293	0.455	77.4	72.0	3.95
5	0.56	142.0	2.73	6690	2.00	0.022	0.055	0.075	0.118	0.192	98.2	93.2	4.61
6	0.55	142.6	2.75	6040	2.25	0.042	0.065	0.101	0.105	0.147	112.2	122.1	4.58
7	0.59	141.5	2.71	7290	2.00	0.057	0.097	0.131	0.147	0.172	79.0	79.6	4.47
Av.	0.55	142.4	2.75	6453	2.19	0.060	0.138	0.190	0.251	0.327	87.7	89.6	4.46

TEST DATA

The physical characteristics of the mixtures, the per cent increase in length and weight, and the per cent of the 28-day strength remaining after a selected number of cycles are shown in Table 4.

In Fig. 2 on the left hand side is shown the variation in compressive strength with cycles of alternate heating and cooling. On the right hand side this same information is shown on a percentage basis.

Linear expansion and gain in weight with increased heating and cooling are shown on a percentage basis in Fig. 3. The effect of using a 50-degree temperature change instead of a 96-degree change is shown in Fig. 4. The character of the surface cracking is shown in Fig. 5.

DISCUSSION OF DATA

Examination of Table 4 and Fig. 2 shows that first there was a sudden reduction in strength caused by the first 50 to 100 cycles of heating and cooling. From this point on, the strength remained about con-

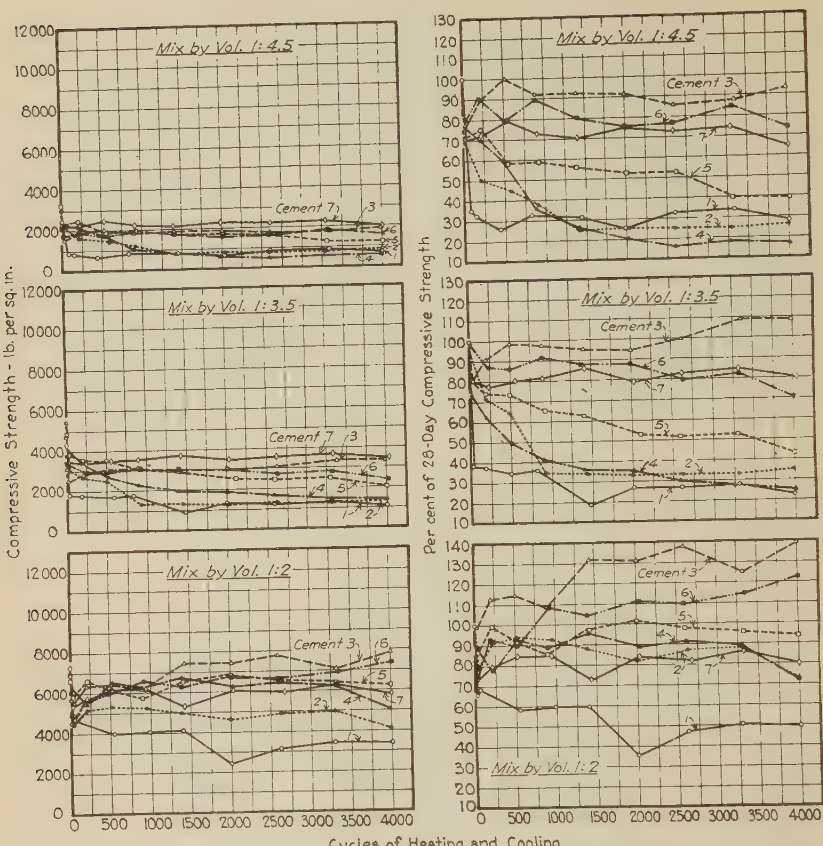

FIG. 2

stant or slightly decreased except for the portland cements No. 3 and No. 6 which in the richer mixes, increased in strength as the test progressed. Cement No. 1 which is apparently the most adversely affected by heating and cooling, is, strangely enough, the most durable in freezing and thawing of any tested in our laboratory. Cement No. 3 in Fig. 2, mix 1:2, shows nearly a normal strength increase and was apparently not seriously affected by the heating and cooling.

Freezing and thawing tests on this cement unfortunately have not been made. There is a general tendency toward an increased per cent of strength remaining after 4000 cycles with increased richness of mix. In this experiment as well as others performed in this laboratory, it appears to be impossible to correlate durability with any of the

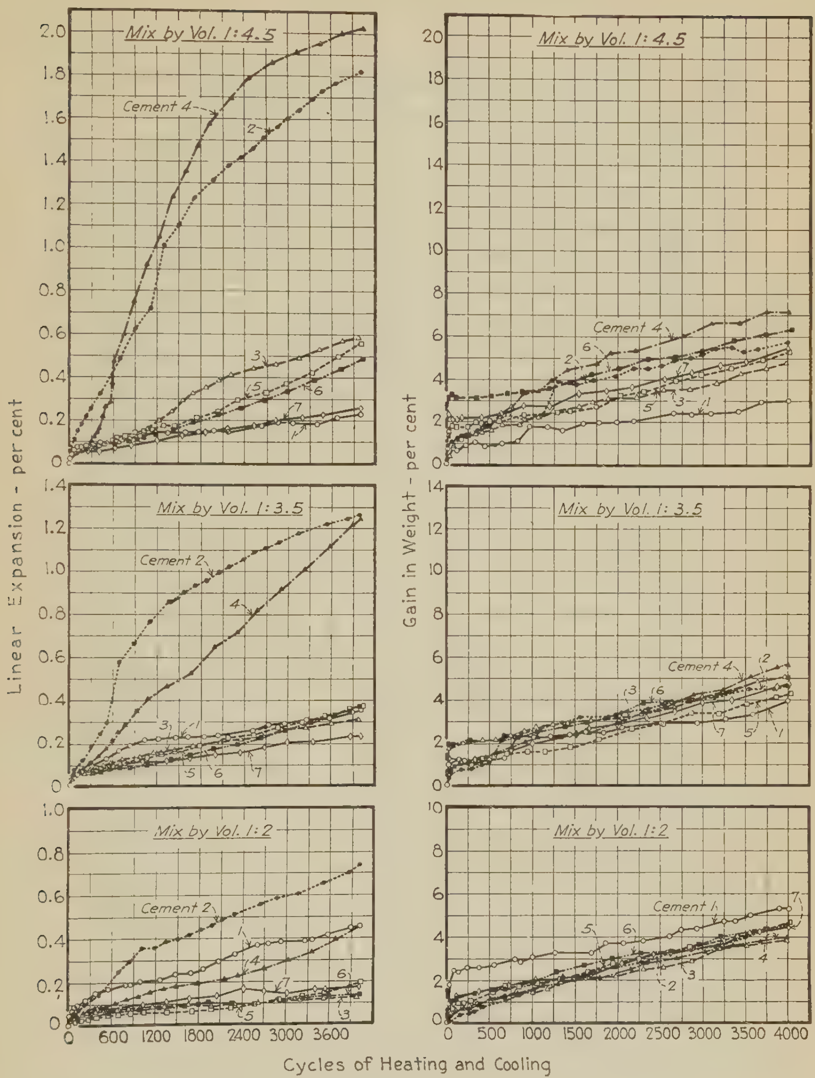

FIG. 3

physical or chemical properties of the cements. It may be of some significance then, that the three most durable mixes were those whose cement had the three highest silica contents.

CHANGE IN LENGTH

Change in length of concrete may be caused by chemical changes during hydration, by changing the temperature or moisture content,

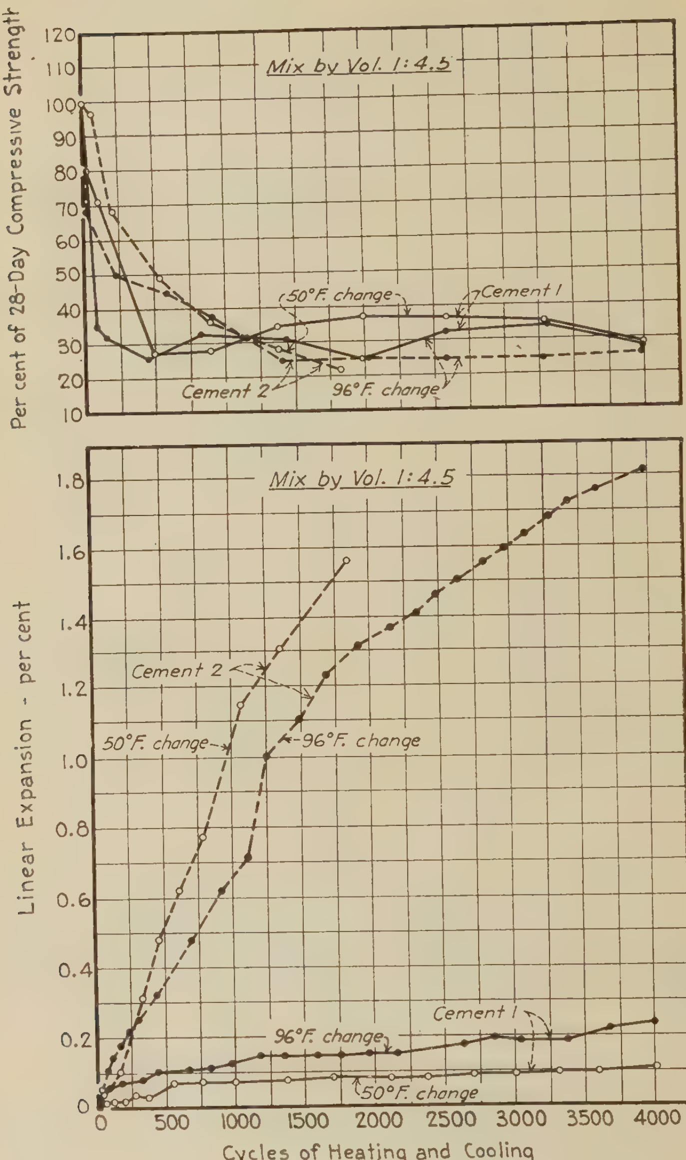
Per cent of 28-Day Compressive Strength
120
110
100
90
80
70
60
50
40
30
20
10
Mix by Vol. 1:4.5
50°F. change
Cement 1
Cement 2
96°F. change
Linear Expansion - per cent
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
Mix by Vol. 1:4.5
Cement 2
50°F. change
96°F. change
Cement 1
96°F. change
50°F. change
0
500
1000
1500
2000
2500
3000
3500
4000
Cycles of Heating and Cooling

FIG. 4

or by applying a steady load. The chemical changes are small after the first few days, and all of the measurements of length in these tests were made on the cold cycle at 68°F. with the specimens saturated with water so that there would be no correction for either temperature or moisture change. However, when concrete is exposed to repeated heating and cooling either above or below the freezing point, it suffers a gradual increase in length. The temperature change of 96°F. used in this test should cause a temporary increase in length of approximately 0.06 per cent. The permanent increases in length actually measured at 4000 cycles and at 68°F. range from 2 to 33 times as great as this. Fig. 3 shows the per cent of permanent increase in length with repeated heating and cooling for the three mixes. With a few exceptions, the per cent increase in length shown on these curves tends to vary inversely with the per cent of 28-day strength remaining after 4000 cycles as shown in Fig. 2. The average per cent increase in length at 4000 cycles for the 1:3½ mix was 0.596 per cent or 7.2 inches per 100 lin. ft. The maximum per cent increase in length after 4000 cycles was found in cement No. 4 for the 1:4½ mix. This percentage of 2.023 is equivalent to 24.3 in. per 100 lin. ft. of concrete. The minimum per cent of increase was 0.124 for cement number 3 in the 1:2 mix which is equivalent to 1.4 in. per 100 lin. ft. of concrete. As noted before, cement number 3 was an ordinary portland cement. The other ordinary portland cement, number 6 in the 1:2 mix, expanded in 4000 cycles an amount equivalent to 1.75 in. per 100 lin. ft.

These figures represent the permanent increase in length at a temperature of 68°F. after 4000 cycles of heating and cooling and to each of them, if the mortar were to be used as pavement in an exposed condition, must be added an additional amount for thermal expansion. To allow for an increase in temperature of 50°F. this additional amount would be about 0.36 in. per 100 lin. ft.

In this investigation as in many others conducted in this laboratory, it seems difficult to correlate any of the physical and chemical properties of the cements with the changes in lengths of the specimens. There may be some significance in the fact that the three mixes having the least change in length were made with cements having the three highest silica contents.

GAIN IN WEIGHT

All specimens in all three mixes gained in weight as the heating and cooling progressed as shown in Fig. 3. Specimens were wiped surface dry with a clean cloth before weighing but in spite of this, after a few hundred cycles, the surface of most of the specimens became coated with a thin film of calcium carbonate. This film was light brown in

color, probably due to the presence of a small percentage of iron. The weight increased suddenly during the first few cycles and then showed a slow constant increase throughout the remainder of the test. In general, the gain in weight varies inversely with the richness of mix. Cement No. 1 is again the exception to the rule, having the least gain in weight in the 1:4½ mix. Cement No. 4 which was seen in Fig. 3,

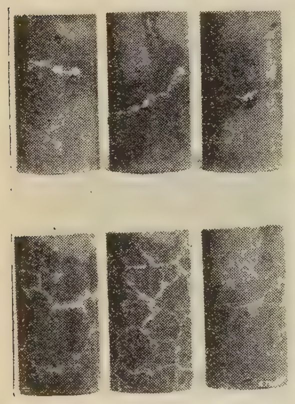

FIG. 5—TYPICAL APPEARANCE OF SPECIMENS AFTER HEATING AND COOLING. CEMENT NO. 5 AFTER 4000 CYCLES OF 96° F. CHANGE IN TEMPERATURE. 1:2 MIX ABOVE; 1:4.50 BELOW

mix 1:4½, to have the greatest increase in length, had the greatest increase in weight, while cement No. 1 had the least change in length, and the least change in weight. This also holds true in the 1:3½ mix but is reversed in the 1:2 mix. The other cements do not indicate a direct relationship between change in length and change in weight. There appears to be an inverse relationship between gain in weight and alumina content and a direct relationship between gain in weight and SO^3 and $CaSO^4$ content.

EFFECT OF A LOWER TEMPERATURE RANGE

Concrete in service is seldom subjected to temperatures as high as 166°F., and since temperatures of 200°F. were definitely known to produce harmful effects, there was a possibility that 166°F. might also be harmful. It was decided therefore, to re-run the test on cements No. 1 and 2 in the 1:4½ mix using a temperature change of 50°F. instead of 96°F. The data from these tests are plotted in Fig. 4. The upper graph of Fig. 4 shows that the 96°F. change in temperature produced a more rapid decrease in strength for the first few hundred cycles. At 1300 cycles there is no difference in the two tests. At 4000 cycles there is little difference between the two tests on cement No. 1. Final results on cement No. 2 are not yet available. The effect of the

two temperature ranges on change in length are shown in Fig. 4, the lower graph. The 96°F. range on cement No. 1 produced about twice the increase in length at 4000 cycles as the 50°F. change. In cement No. 2 the 50°F. change caused 23 per cent more increase in length at 1800 cycles than the 96°F. change did. Fig. 4 apparently indicates that there is little difference in effect between the 50°F. and the 90°F. range in temperatures.

CONCLUSIONS

Mortar 28 days old when repeatedly heated and cooled through a temperature range of 96°F. while submerged in water, is reduced in compressive strength, permanently increased in length, and increased in weight.

A characteristic surface change is always present; first, a checking which increases to deep cracks as the test progresses; these cracks are usually followed by ridges of calcium carbonate.

The compressive strength is reduced more rapidly during the first few alternations of heating and cooling and a considerable amount of damage is done before there is any visible cracking. A temperature range of 50°F. does not reduce the strength as rapidly as a change of 96°F. during the first few hundred cycles, but the effects of the two ranges of temperature were identical at 1300 cycles.

Alternate heating and cooling reduces the compressive strength and increases the change in length of lean mixes to a greater extent than it does rich mixes.

The compressive strengths of specimens made with the two ordinary portland cements and the one special cement were less affected by heating and cooling than were the strengths of specimens made with four early strength cements.

Alternate heating and cooling produced a permanent increase in length from two to thirty-three times as great as the expansion caused by a temperature change of 96°F. Alternate heating and cooling causes mortar to gain in weight, suddenly at first then more gradually the magnitude of the gain in weight varying in general inversely as the richness of mix.

CONCLUSIONS FROM PRELIMINARY TESTS NOT REPORTED IN THIS PAPER

The Kansas River sand used as aggregate in this heating and cooling investigation while meeting specifications for use in concrete and widely used as aggregate in concrete in this part of Kansas, produces less durable concrete than sand from Blue Rapids, Kansas; Holliday, Kansas; Elgin, Illinois, and Cowe Bay, New York.

Concrete having crushed rock for coarse aggregate is less affected by alternate heating and cooling than that having Kansas River sand as aggregate.

Freezing and thawing concrete from 10°F. below zero to 70°F. above zero is more detrimental than heating and cooling from 70°F. to 166°F., and there appears to be a fundamental difference between the character of the action of the two tests. A 3 by 6-in. concrete cylinder may be damaged by freezing and thawing to the extent that the ½-in. stainless steel plugs inserted in the ends for measurement of change in length, either fall out or become too loose to use. The number of cycles of freezing and thawing required to produce this effect varies from 10 to 1600, depending upon the kind of cement used. A characteristic effect of the freezing and thawing test is the rounding of the corners of the specimens giving a bullet-like shape to the ends. The heating and cooling tests although run to 4000 cycles did not round the corners or loosen the screws, the damage appearing to be more severe near the middle of specimens than at the ends.

For the same mix, water cement ratio, and aggregate the durability of the concrete as measured by heating and cooling, and freezing and thawing tests varies considerably with the brand of cement.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.*

Studies of High Pressure Steam Curing of Concrete Slabs and Beams*

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INTRODUCTION

Although the curing of concrete in high pressure steam has been studied from time to time during the last 25 years by various investigators it has not been used commercially except to a very limited extent for a number of years. A review of the available data indicated that more information was needed to explain some of the inconsistencies in practice and to provide a guide for a wider application in the manufacture of concrete products. With this objective a series of studies was made during the last 3 years by the Research Laboratory of the Portland Cement Association, Chicago. The results of these studies have been reported in two papers thus far in the Journal of the Institute and this third paper concludes the report on the investigation.

The studies were begun with small specimens of neat cement or mortar—2-in. cubes and 1 x 1 x 10-in. bars—which were steam cured in a cylinder with an internal diameter of 11 in. and 18 in. high. With these small specimens information was obtained on most of the factors that might be expected to have a bearing on the properties of steam-cured mortar such as the temperature and duration of the steaming period, the age of the specimen before steaming, type and grading of aggregate, type and amount of cement, water-cement ratio, and the influence of various admixtures. The effect of the various factors on the strength, volume change, resistance to freezing and thawing, susceptibility to leaching and efflorescence, and resistance to sodium and magnesium sulfate was determined and the results reported in the Journal for Nov.-Dec., 1934.‡

Following this preliminary series a large steaming cylinder 30 in. in diameter and 10 ft. long was built so that the principal conclusions

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‡Strength and Volume Change of Steam-Cured Portland Cement Mortar and Concrete, by Carl A. Menzel, Amer. Concrete Inst., *Proceedings*, Vol. 31, p. 125.

from the small scale studies could be tested on concrete products of commercial size and something could be learned as to the practical limits of heating and cooling products of different types. With this large cylinder the first studies were confined to tamped hollow concrete block and the results were presented in the second report published in the JOURNAL for Sept.-Oct., 1935.** The next series of experiments went into the field of solid slabs of plain concrete such as might be used in the manufacture of cast stone and a final series developed some information on the bond resistance of steel bars embedded in steam-cured and moist-cured concrete. This paper presents the results of both of these later series of tests and includes the results of recent tests of blocks tested at the age of 1 year which formed the companion specimens of blocks tested 7 days after steaming and which formed the basis of the second paper published in the JOURNAL for Sept.-Oct., 1935.

GENERAL DESCRIPTION

The previous studies on 2-in. mortar cube specimens showed that the useful properties of steam-cured concrete could be developed during exposure for a full 8 hr. to saturated steam at 350° F. and a pressure of about 120 p. s. i. In these studies on small specimens heating in the steaming chamber from 70° F. to 350° F. was gradual over a 5-hr. period. After exposure to steam at 350° F. for the time desired the specimens were permitted to cool gradually for 5 hr. to about 200° F. before they were removed from the steaming chamber.

Sensitive thermocouples indicated that during heating and cooling, the temperature at the center of the 2-in. cubes was practically identical with that of the surrounding saturated steam. Apparently these small specimens were cured under conditions which tended to minimize the development of stresses that can be expected to result directly and indirectly from unequal heating, rate of hardening and cooling.

It appeared that shorter periods than 5 hr. for the initial heating an final cooling could have been used in steaming these small 2-in. cube specimens. It appeared also that the cross-sectional thickness of the solid concrete in a concrete product may be at least 2 in. thick without jeopardizing uniformity of temperature and of curing effect throughout the mass of concrete when cured in a similar manner. However, with larger specimens having a cross-sectional thickness of solid concrete of 6 in. or more it was practically certain that 5-hr. periods of initial heating and final cooling would damage the product and produce erratic results. Obviously, more moderate heating and cooling rates are essential with thick specimens in order to obtain the quality of

**Studies of High Pressure Steam Curing of Tamped Hollow Concrete Block, by Carl A. Menzel, Amer. Concrete Inst., *Proceedings*, Vol. 32, p. 51.

product desired but these rates should not be more moderate than necessary, as long heating and cooling periods reduce plant capacity and increase the cost of steam curing.

A series of studies of rate of heating and cooling was necessary, therefore, to provide definite information as to how fast various products may be heated and cooled without jeopardizing quality and uniformity, and how long the temperature of the steaming chamber must be maintained constant at 350° F. to insure that the concrete in the center of the thickest section had been exposed to 350° F. for a full 8 hr. Obviously some of the major factors in this series are the size and shape of specimen, the thermal properties of the concrete and the rate of temperature rise and fall in the steaming chamber.

The thermal properties of the concrete may be expected to be influenced by such factors as the type and grading of aggregate, the richness of the mix and its consistency, and whether or not the concrete is in a plastic or set condition when steamed. Although products are made from concrete having a wide range of properties, due to some of these factors, it appears that in general the more massive products such as cast stone, precast floor and wall sections will be made with a rich, dry concrete with densely graded aggregates ranging from the heavy to the light-weight types, compacted by vibration or tamping. It is likely also that the practical manufacture of large products may involve at least two plans of operation. Either the product will be molded and permitted to harden 24 or 48-hr. before removal from the molds to the steaming chamber, or will be steam cured in metal molds within three to five hr. after molding.

The above considerations, as to probable type of concrete and manufacturing operations, provided a basis for simplifying the proposed studies with large specimens. Accordingly, a large portion of the information desired on the effect of heating and cooling rates was obtained from studies of specimens 15 in. square made 4, 8 and 12 in. thick with two widely different types of aggregate, (sand and gravel and Haydite) each with one grading, mix and consistency and one method of placement. These specimens were usually steamed at the age of 48 hr. at 350° F. with various periods of heating and cooling. The results of these studies were then checked and augmented by a more comprehensive series of tests on plain concrete beams 4 and 8 in. deep, 12 in. wide, and 36 in. long, using different aggregates, various cement-silica mixtures, and steamed at different ages. The steaming apparatus used was that described in the report on "Studies of High Pressure Steam Curing of Tamped Hollow Concrete Block" referred to previously.

Concrete Materials, Proportioning and Placing

The aggregate grading and concrete proportions for the beam studies are given in Table 7. The 15-in. square specimens for the studies of rates of heating and cracking and checking were of Elgin sand and gravel and Haydite of the same mixes as shown in Table 7 for the beam tests with corresponding aggregates.

The specimens for all series were cast in steel molds. The molds were placed on the vibrating table and 2 or 3 in. of concrete placed on the bottom. Vibration was then started and concrete added as the vibration proceeded until the molds were filled and screeded to uniform surface.

DISCUSSION OF RESULTS

Time Required to Attain Desired Temperature

Fig. 1 gives the time required to heat the interior of concrete specimens 15 in. square of different thickness and type of aggregate to 350° F. under different rates of heating. Heat was applied to raise the temperature of the chamber from 70 to 350° F. at predetermined rates, the temperature then being maintained constant at 350° F. The graphs show that the period required to heat the interior to 350° F. increased with the thickness of the specimens at all rates of heating. They also indicate the probable range of influence of type of aggregate, as the time required to heat the interior of 8 and 12-in. specimens averaged about 22 per cent longer with Haydite than with sand and gravel. For the specimens 4 in. thick, no differences in heating time due to differences in aggregates were observed.

Some tests in which both sand-gravel and Haydite specimens were steamed in the mold at the age of 4 hr. indicated about the same heating time as when steamed after hardening for 48 hr. It is believed, therefore, that the data in Fig. 1 may be applied to the steaming of plastic as well as hardened concrete.

Relation Between Steaming Treatment, Cracking and Checking

Table 1 summarizes the results of tests to determine the effect of different steaming treatments on the cracking and checking of specimens of different thickness. By wetting the smooth surface which resulted from the vibration in steel molds and observing it under strong reflected light it was possible to detect exceedingly fine checks or cracks and thus to note the relative influence of variations in the heating, cooling or pressure release periods of the steaming cycle.

It will be apparent from Table 1 that sand and gravel specimens 4, 8 or 12 in. thick were steam cured in some runs without the development of cracks or checks which could be detected. When the steaming treatment was modified, however, either by decreasing the heating

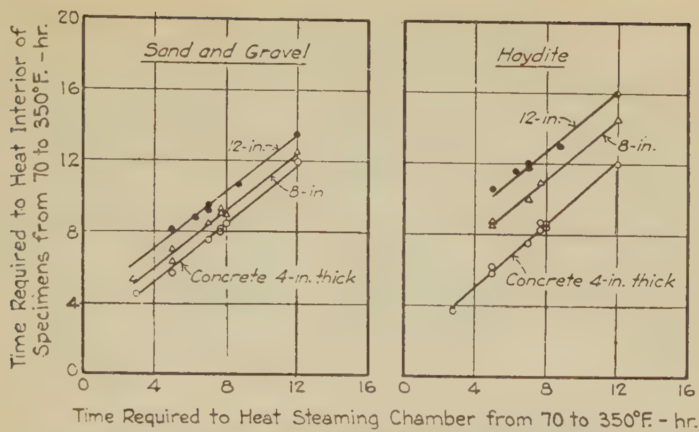

FIG. 1—RELATIONSHIP BETWEEN THE TIMES REQUIRED TO HEAT STEAMING CHAMBER AND INTERIOR OF CONCRETE SPECIMENS TO 350° F.

All specimens 15 in. square; thickness as shown—Chamber temperatures of 212° F. were attained at 70, 110, 165 and 290 minutes respectively with heating periods of 3, 5, 8 and 12 hours. During balance of period of heating to 350° F., the cylinder pressure increased at an approximately constant rate from 0 to 120 lb. per sq. in. gage.

period, the cooling period, or the pressure release period, or by removing the specimens at higher temperatures, checks or cracks were eventually formed which should not be tolerated in high quality concrete. The data also reveal quite clearly that the susceptibility to damage by cracking or checking increased markedly with the thickness of specimen and that a steaming treatment which is satisfactory for one thickness may have to be modified substantially to produce equally satisfactory results with a greater thickness.

The data of Table 1 also show that all Haydite specimens contained checks regardless of the steaming treatment or thickness of specimen and that all 8 and 12-in. specimens contained cracks. On the other hand, although all 4-in. specimens contained checks, none appeared to be cracked. It should be emphasized that the checks present could only be seen when the surface was wetted and observed under reflected light and that they were not believed to be particularly undesirable except possibly in surfaces exposed to the weather. The cracks present in all 8 and 12-in. Haydite specimens were judged to be very undesirable and to impair the usefulness of the concrete for most purposes.

Recommended Minimum Requirements for Steaming to Avoid Cracks or Checks

On the basis of various comparisons which can be made of the data in Table 1, Table 2 and the accompanying notes have been prepared to define the minimum requirements to be observed in steaming concrete

TABLE 1—EFFECT OF STEAMING TREATMENT AND THICKNESS ON CRACKING AND CHECKING OF GRAVEL AND HAYDITE CONCRETE

Specimens 15 in. square and 4, 8 and 12 in. thick made with sand and gravel and Haydite aggregate with grading and mix as detailed in Table 7 were steamed at the age of 48 hr.

Chamber temperatures of 212° F. were attained at 110, 165 and 290 min. respectively with heating periods of 5, 8 and 12 hr. During the balance of the heating period to 350° F. the chamber pressure increased at an approximately constant rate from 0 to 120 lb. per sq. in. gage.

The constant temperature period at 350° F. varied from 12 to 16 hr. and averaged about 14 hr. for all runs. The concrete in the interior of even the thickest specimens was exposed to 350° F. for at least 8 hr.

In the runs with pressure release periods of 5 hr. the pressure decreased gradually with normal cooling of the chamber in conformity with the time-pressure schedule specified under "Cooling Period" in the notes accompanying Table 2. In Run No. 32 with the 4-hr. pressure release period, the pressure decreased as in the 5-hr. period except that the 5-lb. pressure remaining in the chamber after 4 hr. was released through valves to 0 lb. gage in about 3 min. With pressure release periods of ⅓ hr. and 2 hr. the steam was released through valves which were adjusted to reduce the pressure at a constant rate from 120 to 0 lb. in the time designated.

Details of Steaming Treatment					Notes on Checking and Cracking of Specimen				
Run No.	Period in Chamber hr.		Temp. of Specimen When Removed From Chamber ° F.	Pressure Release Period hr.	Thickness of Spec. in.	Gravel		Haydite	
	Heating	Cooling				Cracks	Checks	Cracks	Checks
27	5	24	100-120	5	4 8	None Trace[1]	None Trace[2]	None Many	Many[3] Med.[4]
26	8	24	100-120	5	4 8 12	None None Few[6]	None Trace[5] Few[7]	None Few[9] Med.	Med.[8] Med.[10] Many[11]
31	12	24	120-140	5	4 8 12	None None None	None None None	None Few Med.	Med. Many[12] Many[13]
28	8	10	180-200	5	4 8	None[14] None	None Trace[15]	None Many[17]	Many[16] Many[18]
32	8	4¼	275-300	4	4 8	Trace[19] Few[20]	Med. Many	—— ——	—— ——
29	8	24	90-110	2	4 8	None Few[21]	None Med.[22]	None Few	Many[23] Many[24]
30	8	28	70-90	⅓	4 8	None Few[26]	Trace[25] Many[27]	None Few	Many[28] Many[29]
33	8	⅓	340, Gravel 290-320, Haydite	⅓	4 8	None Few	Many Many	None Many	Many Many

NOTES FOR TABLE 1

(1) None on face but trace at edges.
(2) Practically negligible. Somewhat more numerous and pronounced than in 8-in. specimen, Run No. 26.
(3) Liberal amount of fine checks more numerous and deeper than in 4-in. specimen, Run No. 26.
(4) Medium amount of fine checks somewhat deeper and wider than in 4-in. specimen this run but about the same as in 8-in. specimen, Run No. 26.
(5) Very faint trace, considered negligible.
(6) Cracks present were very fine and continuous.
(7) Enough to be considered undesirable in high quality concrete.
(8) Medium amount fine checks.
(9) Contained some cracks not as bad as 8-in. specimen, Run No. 27.
(10) Medium amount of fine checks somewhat deeper and wider than in 4-in. specimen this run.
(11) Checks somewhat deeper and closer together than in 8-in. specimen this run.
(12) Checks were more numerous and somewhat wider and deeper than in 4-in. specimen this run.
(13) Checks were similar to those in 8-in. specimen this run but slightly deeper.
(14) This 4-in. specimen appears to be equivalent in quality to 4-in. specimen Run No. 26.
(15) Checks were slightly more pronounced than in 8-in. specimen Run No. 26 but definitely less in amount, depth and width than in 8-in. specimen, Run No. 27.
(16) Checks were definitely deeper and more numerous than in 8-in. specimen this run and slightly more pronounced than in 4-in. specimen, Run No. 26.
(17) Cracking was somewhat less in amount and width than in 8-in. specimen, Run No. 27.

(Notes for Table 1, continued on next page)

(Notes for Table 1—concluded)

(18) Checks were not as deep or numerous as in 4-in. specimen this run but were slightly deeper and more numerous than in 8-in. specimen, Run No. 26.
(19) The trace of cracks present appeared to be on verge of becoming longer and deeper. Checks present should be avoided in high quality concrete.
(20) The few cracks present were on verge of becoming more pronounced and numerous.
(21) Cracks very thin, believed to be on verge of becoming wider and deeper.
(22) Although moderate in number the checks present appeared fairly deep.
(23) Checks were fairly deep and wide.
(24) Checks were about equivalent to those in 4-in. specimen this run.
(25) The trace of checks present was judged to be undesirable in high quality concrete.
(26) The cracks present were very fine and appeared to be continuous and on the verge of becoming more pronounced. Should never be tolerated in high quality concrete.
(27) The checks were numerous and well defined. They were more pronounced than in 8-in. specimen, Run. No. 29
(28) The checks were numerous and fairly deep.
(29) The checks were not quite as numerous as in the 4-in. specimen this run but were somewhat more numerous and deeper than in the 8-in. specimen, Run No. 29.

TABLE 2—RECOMMENDED MINIMUM HEATING, COOLING AND CONSTANT TEMPERATURE PERIODS

(For use with sand and gravel and other dense aggregates. See text for discussion of light-weight aggregate)

Heating Period. The flow of steam to the steaming chamber should be adjusted so as to gradually attain chamber temperatures of 212° F. in a period not less than 40 per cent of the total period of heating to 350° F. of 5, 8, or 12 hr. During this first part of the period of heating to 350° F. the chamber should be vented at the top to expel air. During the balance of the period of heating to 350° F. the flow of steam should be adjusted to increase the chamber pressure at an approximately constant rate from 0 to 120 p. s. i. gage.

Constant Temperature Period. During this period the gage pressure should be held as nearly constant as possible at 120 p. s. i. The length of the periods given are based on the graphs in Fig. 1 and will usually be adequate to expose the interior of the specimens to 350° F. for at least 8 hr.

Cooling Period. Specimens 4 and 8 in. thick should not be removed from the steaming chamber at temperatures higher than 180° F. and specimens 12 in. thick at temperatures not higher than 150° F. During each cooling period the pressure should be reduced gradually from 120 to 0 p.s.i. gage in not less than 5 hr. and not faster than according to the following time-pressure schedule: ½ hr., 90 lb.; 1 hr., 70 lb.; 1½ hr., 50 lb.; 2 hr., 40 lb.; 2½ hr., 30 lb.; 3 hr., 20 lb.; 3½ hr., 10 lb.; 4 hr., 5 lb.; 5 hr., 0 lb.

Thickness of Concrete in.	Heating Period to 350° F. hr.	Period of Constant Temperature at 350° F. hr.	Cooling Period From 350° F. hr.	Total Time in Chamber hr.
4	5	9	10	24
8	8	10	12	30
12	12	12	24	48

made with sand and gravel or similar dense aggregates to prevent checking or cracking.

The recommendations embodied in Table 2 may also be applied to the steaming of concrete specimens 4 in. thick made with Haydite, or other light-weight porous aggregates, but as mentioned above some fine surface checks will probably develop. Similar Haydite specimens 8 or 12 in. thick will probably be cracked under the treatment recommended for dense aggregate of the same thickness. Thus far no method of steaming can be recommended which will eliminate surface checking entirely from 4-in. specimens, or formation of surface checks and cracks in 8-in. and 12-in. specimens of Haydite, of the type employed in these tests. Perhaps Haydite having a denser structure and lower absorptive properties than that used could be found which would produce concrete that, like the cinder aggregate concrete used in some tests, would not crack upon steaming.

TABLE 3—FLEXURAL STRENGTH OF PLAIN CONCRETE BEAMS GIVEN DIFFERENT STEAMING TREATMENTS

Beams 12 in. wide, 36 in. long, and 4 and 8 in. deep, made with Elgin sand and gravel graded 0-¾ in. and a cement-silica mixture consisting of 60% by weight of portland cement (Laboratory mixture of 4 brands) and 40% of 0-No. 200 ground Ottawa silica sand.

The Elgin aggregate was graded as follows: 14% by weight, 0-No. 28; 10% No. 28-14; 8% No. 14-8 8% No. 8-4; 30% No. 4-⅜ in.; and 30% ⅜-¾ in.

The mix consisted of 1 part by weight of cement-silica mixture, 5 parts of sand and gravel and a minimum of water required for good placement by high frequency vibration. The concrete contained approximately 635 lb. of cement-silica mixture and 32.4 gal. of water per cu. yd.

The modulus of rupture of each beam was first determined by ⅓-point loading with a span of 34 in. and then by center loading of the longest portion of the broken beam, using a span of 17 in. Three beams were tested for each condition.

Compressive strengths are based on 3 to 6 tests of portions of the broken beams loaded as modified prisms 12 in. high by applying load to the smooth sides of the beams through steel bearing plates directly opposite each other similar to the modified cube tests described by Koenitzer. (L. H. Koenitzer —Proposed Method of Making Compression Tests on Portions of Concrete Beams from Flexure Tests; *Proc. Am. Soc. Testing Mat.*, Vol. 34, Part 2, p. 406-413). With 4-in. beams the effective cross-section of the modified prisms was about 4 by 6 in. and with 8-in. beams about 5 by 8 in.

Run No.	Details of Steaming Treatment: Period in Chamber hr.		Period at Constant Temp. of 350° F. hr.	Temp. of Spec. When Removed From Chamber	Age When Steamed, hr.	Modulus of Rupture p.s.i.: ⅓-Point Loading			Center Loading			Comp. Str. p.s.i.	Notes on Cracking and Checking	
	Heating	Cooling				Min.	Max.	Av.	Min.	Max.	Av.		Cracks	Checks
Beams 4 in. Deep														
38	5	⅓	9½	340	4 48	800 775	855 875	835 820	1020 1010	1195 1135	1130 1075	5910 6195	None Incip.	Many Many
37	5	10	9	180-200	4 48	880 920	960 1010	910 970	1145 1100	1160 1250	1155 1175	6235 6250	None None	None Trace
43	5	30	9	100	4 24 48	905 940 975	970 970 1025	945 955 980	1115 1040 1040	1210 1175 1185	1160 1105 1115	6395 5925 6190	None None None	None None None
39	3	10	9	180-200	4 48	750 880	830 960	800 915	955 1065	1065 1195	1015 1115	5935 6120	None None	Trace Med.
Beams 8 in. Deep														
36	8	⅓	11	340	4 48	655 615	720 750	690 695	855 850	955 1030	915 940	6645 6390	Incip. Incip.	Med. Med.
35	8	12	10	180-200	4 48	865 815	935 830	905 825	1155 1150	1165 1190	1160 1170	6840 6600	None None	Trace Trace
42	5	12	11	180-200	4 48	835 730	855 755	850 740	1050 960	1070 1090	1060 1030	6245 6020	Trace Trace	Med. Med.

Relation Between Steaming Treatment and Flexural Strength of Plain Concrete Beams

Table 3 summarizes the results of tests to determine the influence of different steaming treatments on the flexural strength of plain concrete beams made with sand and gravel aggregate. These tests provided a means for checking the correctness of the recommendations embodied in Table 2. They also served as a means of correlating the qualitative observations on the cracking and checking of the 15-in.

square specimens 4 and 8 in. thick, with quantitative data on moduli of rupture of beams 12 in. wide, 36 in. long and 4 and 8 in. deep.

A study of Table 3 reveals the marked influence of the steaming treatment on the moduli of rupture of beams 4 and 8 in. deep. In this table Run No. 37 and Run No. 35 represent the application of the minimum requirements specified in Table 2 for the 4 and 8-in. beams respectively. When the steaming treatment was altered, as in the other runs represented, either by reducing the heating or the cooling period, lower moduli of rupture were obtained in every instance in both $\frac{1}{3}$ point and center loading tests. It is interesting to note that when the beams were steamed as recommended (Runs No. 37 and 35) the average moduli of rupture of the 4 and 8-in. beams were practically identical for the center loading and very nearly alike for the third-point loading. Comparison of the moduli of rupture of 8-in. beams steamed in Run No. 36 with those of 4-in. beams steamed in Run No. 38, indicate that thick beams are more susceptible to damage by rapid cooling and pressure release than thin beams. Neither the tests in Run No. 43 nor those in any of the other runs indicated any marked influence of age of specimen at the beginning of the steaming cycle on the modulus of rupture. In general, it appears that factors which influenced the modulus of rupture also influenced the compressive strength in a similar manner but to a lesser degree. This indicates that the modulus of rupture is probably a more reliable criterion of the quality of steam-cured concrete than compressive strength. On the whole, there appeared to be excellent correlation between the effects of different steaming treatments on the quality of concrete as judged by observations on the extent and nature of checks and cracks and by the beam strengths.

Effect of Type of Aggregate on Flexural Strength of Steam-Cured Plain Concrete Beams

The results in Table 4 indicate the marked influence of type of aggregate on flexural (and compressive) strength of beams 4 and 8 in. deep when steamed under identical conditions at the age of 48 hr. In general, the moduli of rupture are slightly higher for the 4-in. than for the 8-in. beams, but the compressive strengths are usually somewhat lower. With Haydite both the modulus of rupture and compressive strength were markedly lower for the 8-in. beams. This large reduction in strength is apparently due to the combined effect of cracks and checks which seem to be inherent in steam-cured concrete of thicker sections made with this type of Haydite aggregate. It is interesting to note that the modulus of rupture of the 8-in. cinder beams, which con-

TABLE 4—EFFECT OF TYPE OF AGGREGATE ON FLEXURAL STRENGTH OF STEAM-CURED PLAIN CONCRETE BEAMS

Beams 12 in. wide, 36 in. long, and 4 and 8 in. deep.

Cinders A, porous uncrushed cinders; Cinders B, dense crushed cinders. See Table 7 for information on cinders and other materials used.

Steaming Treatment: At age of 48 hr. the 4 and 8-in. beams were steamed as detailed in Table 3 for Runs No. 37 and 35, respectively, and in conformance with the minimum requirements specified in Table 2 for each thickness.

Modulus of rupture and compressive strength determined at ages of 7 days and 10 months after steaming, as described in notes accompanying Table 3.

Run No.	Type of Aggregate	Age Tested After Steaming	Modulus of Rupture, p.s.i.						Compressive Strength, p.s.i.	Notes on Cracking and Checking	
			⅓-Point Loading			Center Loading					
			Min.	Max.	Av.	Min.	Max.	Av.		Cracks	Checks
					Beams 4 in. Deep						
37	Gravel	7d.	920	1010	970	1100	1250	1175	6250	None	Trace
48	Gravel	10mo.	1030	1065	1040	1035	1275	1190	6590	None	Trace
49	Marble	7d.	575	615	605	660	690	680	4870	None	None
49	Marble	10mo.	630	700	665	770	825	790	5690	None	None
47	Granite	7d.	620	705	675	825	860	835	6230	None	None
47	Granite	10mo.	735	785	750	920	1030	970	6630	None	None
40	Haydite*	7d.	430	505	475	605	675	640	4270	None	Many
45	Haydite	10mo.	500	535	520	635	765	720	4040	None	Many
40	Haydite	7d.	(420)	(485)	(450)	(550)	(660)	(610)	(4170)	(None)	(Many)
67	Cinders A	7d.	415	445	430	525	580	545	2320	None	Trace
67	Cinders B	7d.	570	675	635	750	860	800	3860	None	Trace
					Beams 8 in. Deep						
35	Gravel	7d.	815	830	825	1150	1190	1170	6600	None	Trace
51	Gravel	10mo.	865	935	895	1090	1180	1145	6540	None	Trace
50	Marble	7d.	575	610	590	675	735	705	4970	None	Trace
50	Marble	10mo.	610	660	630	765	770	765	5150	None	Trace
48	Granite	7d.	610	655	635	740	895	795	6340	None	Few
48	Granite	10mo.	665	700	685	885	985	925	7160	None	Few
51	Haydite	7d.	195	220	210	260	285	275	3525	Many	Many
54	Haydite	10mo.	265	385	305	415	505	470	3690	Many	Many
56	Cinders A	7d.	345	400	380	460	490	480	1920	None	Trace
66	Cinders B	7d.	680	680	680	850	865	855	4390	None	Trace

*Values in parentheses are for beams steamed at age of 4 hr. instead of 48 hr.

tained no cracks and only a trace of checks, was appreciably higher than that of the 8-in. Haydite beams having nearly twice the compressive strength but containing cracks and checks. In general, it appears that with the exception of Haydite, the different aggregates represented in Table 4 may be steamed as recommended in Table 2 without the development of cracks or checks. Since the tests with gravel (Table 3) indicated no appreciable difference between beams steamed at ages of 4, 24 or 48 hr. the tests with marble, granite, and cinders were confined to steaming at the age of 48 hr. In one of the runs where 4-in. Haydite beams were steamed at the age of 4 hr. as

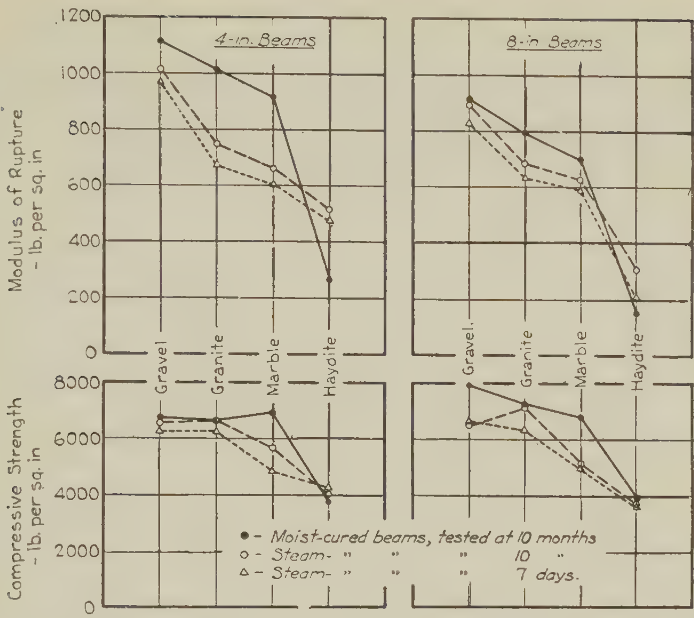

FIG. 2—COMPARISON OF FLEXURAL AND COMPRESSIVE STRENGTH OF STEAM-CURED AND MOIST-CURED CONCRETE

Modulus of Rupture (⅓ point loading) of beams 12 in. wide, 36 in. long and 4 and 8 in. deep. Each value is the average of 3 specimens.
Compressive Strength determined on broken sections. Each value is the average of 6 tests.
Data from Tables 4 and 5.

well as at 48 hr. similar strengths were obtained thus providing further evidence that the age when steaming begins is not important.

Strength Tests of Concrete 7 Days and 10 Months After Steaming

Table 4 includes the results of tests of modulus of rupture and compressive strength made on companion beams at 7 days and 10 months after steaming. These data indicate, in general, that the flexural and compressive strengths of the concrete were at least as great 10 months after steaming as a few days after steaming and that the very great acceleration in strength that occurs during properly applied steam curing provides concrete with substantially permanent strength properties.

Comparison of Flexural and Compressive Strength of Steam-Cured and Moist-Cured Concrete

The graphs in Fig. 2, based on the data of Tables 4 and 5, show quite clearly that the steam-cured concrete made with dense, natural aggregates and the optimum percentages of silica had lower transverse and compressive strengths than corresponding concretes made without

TABLE 5—TESTS OF MOIST-CURED PLAIN CONCRETE BEAMS MADE WITH DIFFERENT AGGREGATES FOR COMPARISON WITH STEAM-CURED BEAMS IN TABLE 4

Beams 12 in. wide, 36 in. long, and 4 and 8 in. deep.
Mix: For a given aggregate the materials and quantities were identical with those used in the steam-cured beams in Table 4 except that straight portland cement instead of the cement-silica mixture given in Table 7 was used. The water used was the minimum required for good placement by high frequency vibration and is also given in Table 7.
Curing: 28 days in moist room and then stored in air of laboratory for 9 months until tested.
Modulus of rupture and compressive strength determined at the age of 10 months as described in notes accompanying Table 3.

Type of Aggregate	Modulus of Rupture, p.s.i.						Compressive Strength, p.s.i.	Notes on Cracking and Checking*	
	⅓-Point Loading			Center Loading					
	Min.	Max.	Av.	Min.	Max.	Av.		Cracks	Checks
Beams 4 in. Deep									
Gravel	1085	1165	1120	1355	1435	1390	6730	None	Many
Marble	915	950	930	1035	1260	1145	6920	None	Many
Granite	1020	1040	1030	1155	1280	1200	6630	None	Many
Haydite	225	325	270	285	335	315	3750	Some	Many
Beams 8 in. Deep									
Gravel	855	965	915	1100	1195	1160	7920	None	Many
Marble	675	730	700	875	930	905	6800	None	Many
Granite	710	840	795	925	995	970	7230	None	Many
Haydite	140	180	155	190	245	215	3920	Many	Many

*Observations for checks and cracks were made in the same manner as described in text for steam-cured specimens under "Relation between Steaming Treatment, Cracking and Checking." Many fine checks were discernible in the smooth moistened surfaces of all 4 and 8-in. specimens, especially with Haydite concrete. Although the gravel, marble and granite specimens were liberally checked they did not appear to contain cracks. However, the 4-in. Haydite specimens contained some fine cracks in addition to the many surface checks, and the 8-in. Haydite specimens contained both fine and large cracks. None of these cracks were readily apparent unless the concrete surface was smooth and moistened. The concrete in the interior of freshly broken Haydite beams was noticeably damp, particularly for the 8-in. specimens.

silica moist cured 28 days and tested at the age of 10 months. The corresponding tests with Haydite concrete, however, showed higher transverse strength for the steam-cured material but about the same compressive strength as for the moist-cured concrete.*

The notes on checking and cracking for Tables 4 and 5 indicate that with dense natural aggregates the surface of the steam-cured concrete contained no checks or only a trace of checks, whereas the surface of the most-cured concrete was liberally checked. Although the surface of Haydite concrete was liberally checked, whether steam-cured or moist-cured, the 4-in. steam-cured specimens did not contain the fine cracks observed in the moist-cured specimens. The surface of the Haydite concrete in 8-in. specimens was liberally checked and cracked whether steam or moist-cured. It appears, therefore, that although steam-cured concrete made with dense natural aggregates may have somewhat lower flexural and compressive strengths than moist-cured

*In this connection it appears important to point out that tamped hollow concrete block having face shells of relatively thin section (1½ in.) in comparison with the solid concrete 4 and 8 in. thick used in these tests, showed compressive strengths at least as great for the steam-cured as for the moist-cured block with Haydite and cinders and substantially higher strengths with sand-gravel aggregate.

TABLE 6—EFFECT OF VARIOUS PERCENTAGES OF 0-NO. 200 SILICA IN THE CEMENT PASTE ON THE FLEXURAL STRENGTH OF STEAM-CURED PLAIN CONCRETE BEAMS

Beams 12 in. wide and 36 in. long, and 4 and 8 in. deep, all made with Elgin sand and gravel graded 0-¾ in. as detailed in Table 7 except that various percentages of silica ranging from 0 to 40% by weight were used.

Steaming treatment: Beams and 6 by 12-in. cylinders were steamed at the age of 48 hr. as detailed in Table 3 for Runs No. 35 and 37 and as recommended in Table 2 for beams of 4 and 8 in. depth.

Modulus of rupture of beams and compressive strength of broken beams (modified prisms) determined as described in notes accompanying Table 3.

Compression cylinders, standard 6 by 12 in., vibrated from the same type of concrete used in the beams.

Run No.	Per Cent Silica By Weight in Cement-Silica Mixture	Modulus of Rupture p.s.i.						Compressive Strength p.s.i.		Modulus of Elasticity of 6 x 12-in. Cylinders p.s.i.
		⅓-Point Loading			Center Loading			5 x 8 x 12-in. Modified Prisms	6 x 12-in. Cylinders	
		Min.	Max.	Av.	Min.	Max.	Av.			
Beams 4 in. Deep										
63	0	1010	1085	1045	1195	1230	1215	6550	4200*	———
63	10	825	905	860	1025	1075	1045	5500	4280*	———
64	20	750	890	835	955	1055	1020	5550	5720*	———
64	30	925	950	935	1045	1130	1060	6170	6400*	———
65	40	895	1015	945	1105	1180	1150	6850	6340*	———
65	50	870	945	910	1025	1155	1090	6380	6100*	———
Beams 8 in. Deep										
54	0	595	680	630	715	860	785	3840	6935**	4,320,000
55	10	730	795	765	885	1035	965	4480	6310**	3,880,000
55	20	835	910	865	1030	1110	1070	5265	6310**	4,160,000
56	30	900	915	910	1080	1245	1160	6045	7235**	4,280,000
35	40	815	830	825	1150	1190	1170	6600	7220**	4,110,000
66	50	805	880	840	1010	1145	1085	6120	6425**	3,400,000

*These cylinders were steamed in Run No. 59 to correspond with treatments for 4-in. beams.
**These cylinders were steamed in Run No. 61 to correspond with treatments for 8-in. beams.

concrete, it will not be as susceptible to surface checking or cracking. Similarly, although Haydite concrete will be checked in 4-in. specimens or checked and cracked in 8-in. specimens with both types of curing, it appears that the steam-cured Haydite concrete is superior from the standpoint of flexural strength.

Effect of Fine Silica on Flexural and Compressive Strength

The curves of Fig. 3 based on the data in Table 6, show the effect of different percentages of fine silica (ground Ottawa Sand 0-No. 200) in the cement paste on the flexural and compressive strength of concrete in 4 and 8-in. beams made with sand and gravel. It will be seen that with 30 or 40 per cent of silica, the modulus of rupture of 8-in. beams was approximately 50 per cent higher than when no fine silica was present, except that supplied by the fine sand particles in the active 0-No. 48 size range. In contrast with these results with 8-in. beams, the tests with 4-in. beams showed no improvement in the modulus of rupture with 30 or 40 per cent of silica over mixes containing nominally

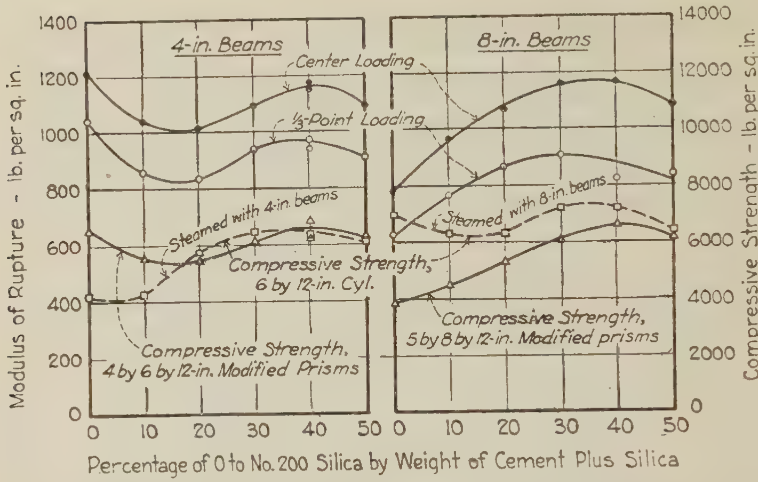

Fig. 3—Effect of various percentages of silica on the flexural strength of steam cured plain concrete beams

Aggregate: Elgin sand and gravel
Specimens tested within 7 days after steaming.
Data from Table 6

0 per cent silica. However, it is interesting to note that with 30 or 40 per cent of silica the flexural strengths of 4 and 8-in. beams were nearly identical even though with nominally 0 per cent silica they were quite different. This difference in influence of silica on flexural strength (and on the compressive strength of the modified prisms) can not be satisfactorily explained without considerable further study, although it is believed mainly to be due to differences in the time of curing and thickness of specimens.

Fig. 3 also shows that different percentages of fine silica appear to have the same general effect on the compressive strength when determined on broken beam sections tested as modified prisms, as on the flexural strength of the original beams. Compressive strengths based on tests of 6 by 12-in. cylinders (dash line curves in Fig. 3) showed considerable variation from the tests on the modified prisms. Again it is believed that this variation reflects the influence of differences in curing that the concrete received due to the differences in the size and shape of the cylinders and beams. This is borne out by the fact that the different steaming treatments used for the 4 and 8-in. beams had a marked influence on the strength of the cylinders, particularly for lower percentages of silica.

In general, the test on large specimens confirms the results obtained with small specimens with respect to the advantage of using fine silica

TABLE 7—WEIGHTS OF MATERIALS AND GRADING OF AGGREGATES USED IN BEAM SPECIMENS

In general, beams were made with a cement-silica mixture consisting of 60% by weight of portland cement (laboratory mixture of 4 brands) and 40% of Ottawa silica sand ground 0-No. 200 sieve. In the case of Haydite beams the cement-silica mixture consisted of 75% of cement and 25% silica.

In all cases the water used was the minimum required for good placement by high frequency vibration.

Aggregate	Per Cent By Weight of Each Sieve Size in 0-¾-in. Aggregate Mixture—Tyler Std. Sieves						Weight of Materials Used lb. per cu. yd. Concrete			
	0-28	28-14	14-8	8-4	4-⅜ in.	⅜-¾ in.	Aggregate (Dry Basis)	Cement-Silica Mixture	Water	Total
Elgin sand & gravel	14.0	10.0	8.0	8.0	30.0	30.0	3180	635	270	4085
Georgia white marble	17.6	5.8	7.4	12.9	45.6	10.7	3150	630	263	4043
Wisconsin red granite	14.2	6.5	10.5	18.8	42.0	8.0	3045	609	279	3933
Western Haydite	21.0	12.0	8.5	8.5	25.0	25.0	1530	634	422	2586
Porous uncrushed cinders, A*	20.0	10.0	8.0	20.0	42.0	0	1840	590	550	2980
Dense crushed cinders, B	20.0	10.0	8.0	20.0	42.0	0	2710	620	410	3740

*Two distinctly different types of cinders were obtained by separating the same lot of soft-coal power plant cinders, ranging from dust to about 1-in. size, on the ⅜-in. screen. One type, a light uncrushed porous cinders, comprised material passing the ⅜-in. screen. The other type consisted of material crushed from the ⅜-in. to 1-in. cinder particles which produced a relatively heavy, dense cinder aggregate having a unit weight about 50% higher than the porous uncrushed cinders.

with steam-cured concrete. They also indicate that the use of silica in optimum percentages (30-40%) in the large specimens reported in Tables 1, 3 and 4 was fully justified regardless of the type of aggregate used.

COMMENTS ON VARIOUS FEATURES OF THE STUDY

Lightness of Color

During the steaming treatment the normally grey color of the cement is bleached to an almost white color. As a result the steam-cured concrete is markedly lighter than similar moist-cured concrete.

Moisture Loss During Steaming

All steam-cured concrete had a lower moisture content after than before steaming. Specimens 4 in. thick made with various aggregates usually lost from 35 to 40 per cent of the total moisture present in the concrete at the time of molding; specimens 8 in. thick lost from 25 to 30 per cent. Most of the moisture lost leaves the specimens during the cooling and pressure-release periods in the steaming chamber.

Whitish Deposit After Steaming

When removed from the steaming chamber the surfaces of the slabs and beams made with rich concrete were covered with a very light, fluffy white deposit which could easily be dusted or wiped off with a damp rag to expose a smooth unblemished surface. This deposit was observed with all types of aggregate used and was found by analysis to be principally calcium carbonate.

TABLE 8—COMPARISON OF BOND RESISTANCE OF STEEL BARS EMBEDDED IN MOIST-CURED AND STEAM-CURED SAND AND GRAVEL CONCRETE

Bond tests made by applying a pull on one end of round steel bars 24 in long embedded axially in 8 by 8-in. concrete cylinders as illustrated in Fig 1, Bulletin 17, Structural Materials Research Laboratory, Lewis Institute, Chicago. Speed of movable head of testing machine was .057 in. per min. and end slip of bar was measured with a .0001-in. Federal gage. With ½-in deformed bars the maximum bond was limited either by stretching of steel or continued slip under load; with 1-in. deformed bars the maximum bond was limited by splitting of cylinders. Readings usually continued until slip of about 0.1-in. for plain bars and 0.2-in. for deformed bars occurred.

Compression tests made on standard 6 by 12-in cylinders from the same concrete and placed in the same manner as the pull-out specimens.

The Elgin aggregate was graded as follows: 14 8% by weight of 0-No. 28; 9.25% No. 28-14; 6.30% No. 14-8; 6.65% No. 8-4; 15.75% No. 4-⅜ in.; 31.5% ⅜-¾ in.; 15.75% ¾-1½ in.

The mix consisted of 1 part by weight of cement (or cement-silica mixture), 7 parts of sand and gravel, and the minimum of water required for good placement by hand rodding and high frequency vibration. The concrete contained approximately 3180 lb. of aggregate and 455 lb. of cement or cement-silica mixture per cu. yd. For hand placing the water used was 31.8 and for vibration 27.6 gal. per cu. yd. Moist-cured specimens were made with both normal (N) and high early (H.E.) strength cements. The normal portland cement was a mixture of equal parts of 4 brands; the high early strength cement a mixture of equal parts of 3 brands. Steam-cured specimens were made with normal strength cement (N) and with a cement silica mixture (C.S.) consisting of 60 per cent by weight of cement (N) and 40 per cent of 0-No. 200 Ottawa silica sand (S).

In molding the pull-out cylinders the embedded bars were accurately centered and held firmly in vertical position during compaction of the concrete. Concrete placed by hand was rodded in 3 layers in standard manner. Concrete placed by vibration was placed continuously in the mold until filled during the first ½ min. of vibration and was vibrated for ½ min. after filling. During vibration the mold was clamped firmly to the vibrating table operated at a frequency of 4200 vibrations per min. Special precautions were taken to prevent leakage of water from the mold, particularly where the bar passed through the hole in the bottom plate of the mold, to avoid honeycomb at this point. Steam-cured specimens were steamed at age of 24 or 48 hr. for 9 hr. in saturated steam at 350° F. with a gradual heating period of 5 hr. and cooling period of 10 hr.

Steel Bars: Plain bars were commercial "hot rolled" steel ½ and 1 in. in diameter; deformed bars were nominally ½ and 1 in. in diameter designated commercially as "new billet deformed round concrete reinforcing bars, intermediate grade." The latter had arc-shaped projections or lugs staggered on 2 opposite sides of the bar. The ½-in. bars had 31 lugs per ft., about $\frac{1}{32}$ in. high, $\frac{1}{16}$ in. wide (axially) and ½ in. long (circumferentially) spaced at a pitch of ¾ in. Similarly the 1-in. bars had 15 lugs per ft., about $\frac{3}{64}$-$\frac{1}{16}$ in. high, $\frac{3}{32}$ in. wide, and 1⅛ in. long, with a pitch of 1⅝ in.

Bond and compressive strengths are the averages for 3 specimens.

Size and Type of Round Bar	Cement	Moist Cured (Tested Damp as Taken From Moist Room): Age at Test, 7 Days: Bond Strength, p.s.i.: At Slip of .0005 in.	Maximum	Ratio, at .0005 in. to Max., %	Compressive Str., p.s.i.	Age at Test, 28 Days: Bond Strength, p.s.i.: At Slip of .0005 in.	Maximum	Ratio, at .0005 in. to Max., %	Compressive Str., p.s.i.	Cement	Steam Cured (Tested Semi-Dry as Taken From Steam Chamber): Bond Strength, p.s.i.: At Slip of .0005 in.	Maximum	Ratio, at .0005 in. to Max., %	Compressive Str., p.s.i.
Placed By Hand Rodding														
½ in. Plain	N.	200	263	76	3630	290	423	68	5240	N.	38	159	24	2960
	H.E.	233	330	70	5720	245	346	71	6670	C.S.	146	319	46	4760
1 in. Plain	N.	180	260	69	3630	297	412	72	5240	N.	39	211	19	2960
	N.E.	274	429	64	5720	293	597	49	6670	C.S.	113	433	26	4760
½ in. Def.	N.	286	1196	24	3630	372	1183	31	5240	N.	106	1184	9	2960
	H.E.	390	1176	33	5720	471	1173	40	6670	C.S.	318	1188	27	4760
1 in. Def.	N.	254	780	33	3630	354	1051	34	5240	N.	70	780	9	2960
	H.E.	379	1170	32	5720	367	1270	29	6670	C.S.	142	1211	12	4760
Placed By High Frequency Vibration														
½ in. Plain	N.	223	340	65	4800	199	417	48	6450	N.	42	192	22	4450
	H.E.	283	445	64	6750	331	509	65	7330	C.S.	115	337	34	6360
1 in. Plain	N.	226	405	56	4800	238	546	44	6450	N.	16	264	6	4450
	H.E.	324	612	53	6750	337	771	44	7330	C.S.	109	432	25	6360
½ in. Def.	N.	319	1151	27	4800	335	1176	29	6450	N.	122	1215	10	4450
	H.E.	541	1171	46	6750	486	1184	41	7330	C.S.	229	1216	19	6360
1 in. Def.	N.	310	989	31	4800	358	1345	27	6450	N.	85	1273	7	4450
	H.E.	435	1279	34	6750	467	1538	30	7330	C.S.	120	1282	9	6360

Constancy of Shape and Dimensions

Tests with steel straight edges and careful measurements showed that under the steam curing the slabs and beams retained their shape and dimensions as well as the character of their surfaces.

Bond Resistance of Reinforcing Steel

Tables 8 and 9 give the results of tests to provide information on the bond developed in pull-out tests of plain and deformed steel bars embedded in both steam and moist-cured concrete. A study of these results indicates in general that the bond stress between steel and steam-cured sand and gravel concrete at a slip of .0005 in. was only 30 to 50 per cent of that developed in the moist-cured concrete regardless of the size or type of bar used. The bond stress between the steel and steam-cured Haydite and cinder concrete at a slip of .0005 in. was as low or lower than with the steam-cured sand and gravel concrete. In view of the generally low bonds developed in these tests it appears that the steam-curing of reinforced concrete products should not be attempted commercially without thorough tests of the suitability of each type of product for the purpose for which it is intended in service.

Modulus of Elasticity

The results of some tests on the modulus of elasticity of steam-cured sand-gravel concrete are included in Table 6. The values given are based on the secant-moduli of stress-strain curves which were nearly straight lines for loads up to about 85 per cent of the ultimate compressive strength. It will be seen that the modulus of elasticity varied from about 3,400,000 to 4,380,000 in an apparently close relationship with compressive strength. The percentage of silica present appears to influence the modulus of elasticity in the same manner as it influenced the compressive strength.

Maximum Size of Aggregate

Tests with sand and gravel and with Haydite graded both 0 to ⅜ in. and 0 to ¾ in. indicated that the size range of the aggregate did not constitute a factor influencing the results reported herein. They showed that the cracking and checking reported for the Haydite specimens could not be altered by reducing the size range. Other tests showed that the checking could not be eliminated by substituting Elgin sand for fine Haydite particles.

ONE-YEAR BLOCK TESTS

Table 10 gives the results of recent tests on the compressive strength of steam-cured block one year after steaming not available at the time the report on the series of block tests was published in the JOURNAL for Sept.-Oct., 1935. These data indicate in general that block stored in

TABLE 9—BOND RESISTANCE OF STEEL BARS EMBEDDED IN STEAM-CURED HAYDITE AND CINDER CONCRETE

Tests and test specimens were made in essentially the same manner as described in the notes accompanying Table 8. All specimens represented in this tabulation were placed by vibration and were steam cured. Unless otherwise noted the maximum bond was limited by continued slip of the steel under load rather than by stretching of the steel or splitting of cylinders.

The Haydite and porous uncrushed cinders were graded as described in Table 7 but were combined with the cement and cement-silica mixtures given below. The portland cement used was the laboratory mixture of equal parts of 4 brands (same as N, Table 8) and the silica was 0-No. 200 Ottawa silica sand (same as S, Table 8). The mix contained the minimum of water required for good placement by high frequency vibration. The quantity of water ranged from 49 to 53 gal. per cu. yd. for the Haydite and from 63 to 66 gal. per cu. yd. for the cinder concrete.

Steel Bars: Same as described in notes accompanying Table 8.

Bond strengths are the averages for 3 specimens, compressive strengths for 6 specimens.

Size and Type of Round Bar	Haydite						Cinders					
	Per Cent Silica By Wt. in Cement-Silica Mixture	Wt. of Cement or Cement-Silica Mixture, lb. per cu. yd.	Bond Strength, p.s.i.			Compressive Str., p.s.i.	Per Cent Silica By Wt. in Cement-Silica Mixture	Wt. of Cement or Cement-Silica Mixture, lb. per cu. yd.	Bond Strength, p.s.i.			Compressive Str., p.s.i.
			At Slip of .0005 in.	Maximum	Ratio, at .0005 in. to Max., %				At Slip of .0005 in.	Maximum	Ratio, at .0005 in. to Max., %	
½ in.	0	370	40	109	37	3000	0	655	35	88	40	1370
Plain	25	370	89	229	39	3200	40	655	87	197	44	2330
Plain	25	616	302	579	52	4280	40	755	77	264	29	3300
1 in.	0	370	29	153	19	3000	0	655	19	64	30	1370
Plain	25	370	47	248	19	3200	40	655	32	130	25	2330
Plain	25	616	62	260	24	4280	40	755	37	195	19	3300
½ in.	0	370	93	917	10	3000	0	655	33	510	6	1370
Def.	25	370	126	1037	12	3200	40	655	46	804	6	2330
Def.	25	616	188	1109	17	4280	40	755	26	1011	3	3300
1 in.	0	370	50	732	7	3000	0	655	20	346	6	1370
Def.	25	370	95	773	12	3200	40	655	60	669	9	2330
Def.	25	616	110	869*	13	4280	40	755	40	914*	4	3300

*Maximum bond limited by splitting of cylinder.

TABLE 10—STRENGTH OF STEAM-CURED BLOCK TESTED 7 DAYS AND 1 YEAR AFTER STEAMING

All block tested in air-dry condition at equilibrium with dry air of laboratory.

Blocks tested 1 year after steaming were companion specimens of block tested 7 days after steaming which formed the basis for the previous paper, Studies of High Pressure Steam Curing of Tamped Hollow Concrete Block, by Carl A. Menzel, in the JOURNAL of the Amer. Concrete Inst., for Sept.-Oct., 1935.

Run No. and Block Group	Type of Aggregate	Compressive Strength, p.s.i.		Run No. and Block Group	Type of Aggregate	Compressive Strength, p.s.i.		Run No. of Block Group	Type and Aggregate	Compressive Strength, p.s.i.	
		7d.	1yr.			7d.	1yr.			7d.	1yr.
5-R	Sand-Gravel	1400	1625	9-X	Sand-Gravel	1445	1490	9-Y	Haydite	750	810
5-R	"	2130	2600	9-X	"	1960	2300	9-Y	"	1200	1200
5-R	"	2515	3200	9-X	"	2700	3060	9-Y	"	1485	1445
5-P	"	1320	1330	7-U	"	1450	1490	11-AD	"	675	710
5-P	"	2100	2220	7-U	"	2240	2490	11-AD	"	975	1005
5-P	"	2710	3200	7-U	"	2760	3100	11-AD	"	1280	1330
6-T	"	1260	1420	8-V	Haydite	660	585	11-AE	Cinders	530	510
6-T	"	2130	2660	8-V	"	1230	1075	11-AE	"	980	1000
6-T	"	2760	3100	8-V	"	1900	1990	11-AE	"	1675	1850
12-AG	"	1320	1515	12-AH	"	760	755	10-AB	"	815	775
12-AG	"	1925	2215	12-AH	"	1100	1250	10-AB	"	1100	990
12-AG	"	2255	2740	12-AH	"	1450	1370	10-AB	"	1400	1340
8-W	"	1530	1985	10-AA	"	870	865				
8-W	"	2700	3200	10-AA	"	1420	1400				

the laboratory for one year after steaming had strength equal to or greater than that developed in tests soon after steaming and that the high early strength developed by steam curing is permanent. The permanence of strength of steam-cured concrete appears therefore to be well established by these tests for characteristically lean tamped concrete as well as for the rich, dense concrete tested in flexure and compression as reported in Table 4 and discussed under "Strength Tests of Concrete 7 Days and 10 Months after Steaming."

SUMMARY

The principal results and recommendations may be summarized as follows:

(1) Plain concrete products, simulating cast stone, were cured successfully in saturated steam at 350° F. during a 24 to 48-hr. period in the steaming chamber, the exact time depending on the thickness of the concrete. The curing of products 4 in. thick was completed in 24 hr., 8 in. thick in 30-36 hr., and 12 in. thick in 42-48 hr.

(2) It was found that the steaming treatment could begin at any convenient time from 4 to 48 hr. or more after molding.

(3) The tests show that in products plants the steaming cycle will need to be varied for different thicknesses of product, but should always provide for a period of constant temperature and pressure with appropriate periods of gradual heating and cooling. During the heating period the temperature in the steaming chamber should rise gradually to 350° F., and during the cooling period the temperature and pressure should be gradually lowered. The proper steaming cycle and recommended minimum requirements for steaming various thicknesses are described in Table 2.

(4) The tests indicate that large specimens made with dense, natural aggregates, such as sand and gravel, granite, and marble, may be steam cured following the recommendations in Table 2 without distortion or the development of cracks or fine checks. The steam-cured concrete was comparatively dry and light in color. By the use of optimum percentages of silica, leaching or efflorescence will be largely eliminated.

(5) For concretes using Haydite, or other light-weight porous aggregates, the recommendations embodied in Table 2 may be applied to the steaming of specimens up to 4 in. in thickness, but some fine surface checks will probably develop. Haydite specimens, 8 or 12 in. thick, will probably be cracked under the treatments recommended in Table 2 for dense aggregate of corresponding thickness. Thus far, no method of steaming of concrete made with Haydite, of the type employed in these tests, can be recommended which will eliminate entirely

surface checking in 4-in. specimens or the formation of surface checks and cracks in 8 or 12-in. specimens.

(6) Good correlation was found between the transverse strengths of beams and the extent and nature of checks and cracks for different steaming treatments. Absence of checks and cracks in a smooth concrete surface is believed to give a very reliable indication of the quality of the steam-cured concrete and of the suitability of the steaming treatment used.

(7) The results with large specimens confirm those obtained with small specimens with respect to the advantage of using fine silica in rich, steam-cured concrete in optimum percentages of 30 to 40 per cent, regardless of the type of aggregate used.

(8) Steam-cured concrete made with dense, natural aggregate and the optimum percentage of silica showed transverse and compressive strengths from 5 to 30 per cent lower than corresponding concretes made without silica but moist cured 28 days and tested at the age of 10 months. Corresponding tests with Haydite concrete showed substantially higher transverse strength for the steam-cured material but about the same compressive strength as for moist-cured concrete.

Although steam-cured concrete made with the denser, natural aggregates had somewhat lower strength it appeared markedly superior with regard to freedom from surface checking and cracking to moist-cured concrete.

(9) Tests conducted at the age of 10 months and 1 year indicate quite definitely that the high early flexural and compressive strength developed by proper steam curing is permanent.

(10) A feature of the tests was the low bond resistance in steam-cured concrete with both plain and deformed bars. The bond stress at an end slip of bar of .0005 in. was only 30 to 50 per cent of that developed in comparable moist-cured concrete. In steam-curing reinforced products special consideration must therefore be given to the low bond strengths developed.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.

Factors of Workability of Portland Cement Concrete*

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1. INTRODUCTION

THIS paper presents some results obtained by several test methods which were designed to measure the "workability" of concrete. They were developed after attempts had been made to use the most promising test methods presented or suggested in the many papers on this and related subjects. The methods described in this paper are not given as fulfilling all requisites of the needed test procedure or as yielding numerical values indicative of workability, but are presented to show how the subject is being studied, and some of the factors which seem to be components of this property. It is hoped that this presentation will be of interest and be productive of comments which will assist in the work.

According to Pearson(1) workability depends on several factors and even the commonly used slump and flow tests, "furnish an indication of only one of the factors of workability (mobility or wetness) * * * " Others have assumed that a single test method could be found, although no single test procedure for the measurement or workability of concrete has been proven entirely satisfactory or been universally accepted. Even though the workability of a concrete may eventually be expressed by a single numerical value, it is desirable to measure each independent factor that affects it. All tests that have been proposed for measuring workability were considered from the standpoint of their ability to measure the separate factors composing this complex quantity but none appeared satisfactory from this aspect.

Any concrete of whatever richness or leanness, or howsoever proportioned, may have its water content adjusted to give any desired flow within reasonable limits, yet concretes at any one arbitrarily

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(1)References will be found at the end of the paper.

selected flow obviously differ in workability characteristics. A rich (1:1:2) concrete possesses a fatness, apparent even to the novice, to a degree that is lacking in a 1:2 ½:5 concrete. Another characteristic is "stickiness" of concrete, a factor of workability suggested by Bates and Dwyer(2). New Tests are therefore needed to show these different characteristics. Preliminary experimental work in this investigation indicated that harshness, segregation, shear resistance and stickiness are factors of workability of concrete, and tests intended to measure these factors were developed.

2. MATERIALS

Potomac River sand and gravel were used as aggregate throughout the studies. A typical grading of the sand was as follows:

Sieve No.	Percentage Retained
4	2.6
8	15.9
16	30.4
30	51.5
50	85.5
100	98.0
F. M. (Fineness modulus)	2.84

The gravel was separated into four sizes; No. 4 to ⅜ in., ⅜ in. to ¾ in., ¾ in. to 1 in. and 1 in. to 1½ in., and recombined into several gradings.

Several brands of cement, with apparently diverse plastic properties, and specific surfaces varying from 1440 to 2100 sq. cm. per gram, were included. Diatomaceous silica was used in some of the tests as an admixture.

3. TEST METHODS

a. Harshness

A simple illustration of a difference in concretes having the same flow* is the rapidity with which they spread or flow on the flow table during the course of 15 drops of the table. The "plasticity coefficient," (ratio of slump to flow) was suggested by Pearson and also Burmister(3) as a measure of harshness in concrete, but this coefficient was not adopted because the slump test values have such poor reproducibility. The "spread" after two drops of the flow table, however, does give a distinct measure of this difference between mixes having the same flow, particularly when the wetter mixes are used.

Fig. 1 presents the spread after two drops, plotted against the flow, of two mixes in which the cement-water ratio by weight (c/w) varies

*As the flow test is made at the National Bureau of Standards, the concrete is stirred (not rodded) into a flow cone having a lower diameter of 10 in., upper diameter of 6¾ in., and height of 5 in. After the mold has been removed, the table is raised and dropped 15 times in 10 seconds, by means of a cam, through a distance of ½ in. The percentage increase in diameter of the concrete is the "flow." Therefore unless otherwise noted, flow will be considered as measured after the table has been dropped 15 times.

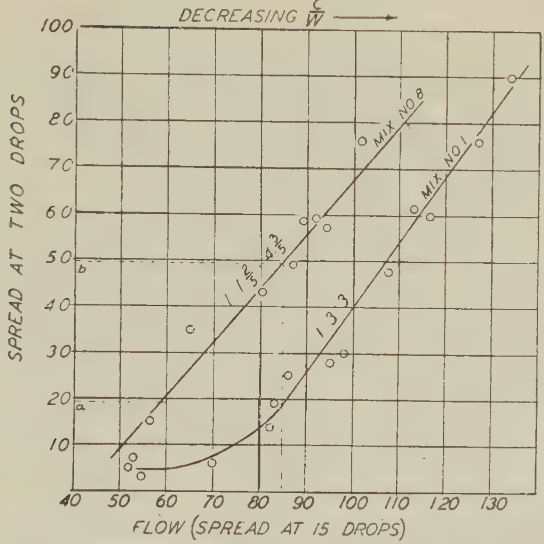

FIG. 1—RELATION BETWEEN SPREAD AFTER 2 DROPS, AND FLOW. "A" INDICATES THE HARSHNESS OF THE 1:3:3 MIX AND "B" INDICATES THE HARSHNESS OF THE 1:1 2/5:4 3/5 MIX

over a wide range. Fig. 2 shows the spread plotted against the number of drops, again using concretes of varying c/w and flow characteristics. It may be seen that in the three comparatively dry concretes, with a flow of 50 to 60, the spreads after two drops were not appreciably different, whereas for concretes having a flow of about 100 the spreads after two drops varied from 25 to over 60.

The spread after two drops, obtained by interpolation from a graph similar to Fig. 1 for a concrete having a flow of 85, was adopted as a measure of harshness. A flow of 85 was selected because a concrete with higher flow might develop a "water ring" on the flow table, while the test would not be sufficiently sensitive with drier mixes.

b. Segregation

Ideal concrete during flow in a chute or into place in the form will be of the same proportions throughout as the original mix after it has reached its final location. Departure from the original proportions, (i. e., segregation) is undesirable. Pearson and Hitchcock(4) state that "workability is the inverse of the degree of effort required in handling and placing concrete in such a manner as to give a uniform and homogeneous finished product * * * " Pearson(1) also states that a test for segregation should supplement the flow test. Accordingly, segregation is a factor in the workability of concrete.

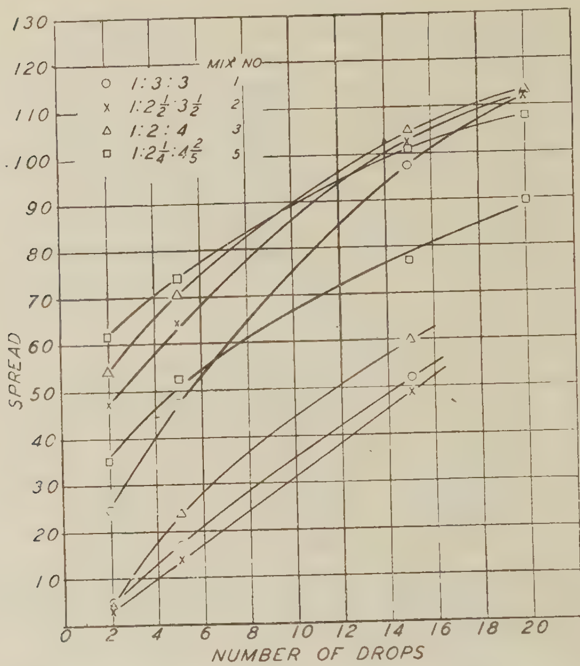

FIG. 2—VARIATION OF SPREAD WITH NUMBER OF DROPS, USING FOUR MIXES AND TWO DIFFERENT C/W RATIOS FOR EACH MIX

Since we cannot hope to eliminate the undesirable property of segregation, but only to hold it at a minimum, a measure of segregation is necessary.

The test procedure which is being used in this work is as follows:

A standard No. 4 sieve, 2 in. high, with 7 in. diameter of sieve exposed and with attached bottom pan, is filled with one-fourth of a batch of concrete, placed (but no fastened) on the flow table and the table dropped ⅛ in. thirty times. After the entire batch has been treated in this manner, the accumulated material in the pan is weighed and recorded. The measure of segregation is taken as the ratio of the weight of the mortar* in the pan to the total mortar in the batch, expressed in per cent of the latter.

c. Shear Resistance

In devising a test for shear resistance, the possibility of segregation during the test should be reduced to a minimum. Yoshida(5) expresses

*The total weights of sand, cement and water, the latter corrected for an absorption of 1 per cent of the weight of the coarse aggregate, are taken as the mortar in the batch.
An absorption correction for the total aggregate was applied throughout in calculating c/w.

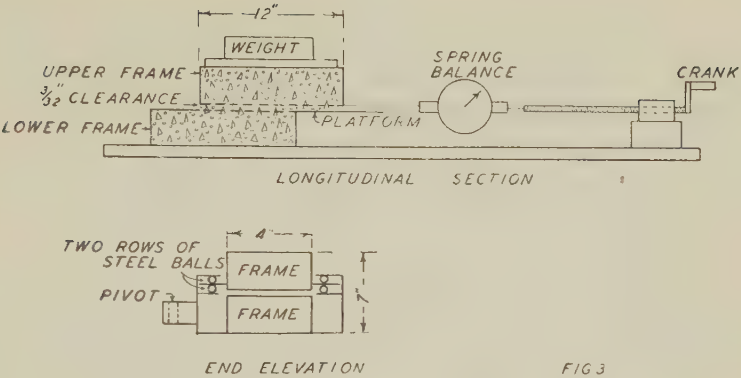

FIG. 3—APPARATUS FOR MEASURING RESISTANCE TO SHEAR, SHOWN SCHEMATICALLY

this in stating that the flow table measurement is open to the criticism that "it is concerned with determining the workability of concrete in a segregated condition," and adds that "with the flow table reliable results may be obtained only with the concrete in which practically no segregation occurs during the process of the test."

In the present investigation a device (Fig. 3) for measuring shear resistance (with a minimum of segregation) has been evolved from the device of Terzaghi(6), developed and used by him for the study of shear resistance of soils. The apparatus used consists essentially of two rectangular frames, each 3½ in. deep by 4 in. by 12 in., the upper frame being supported on ball bearings attached to the lower frame and in such a way that there is a clearance of $\frac{3}{32}$ in. between the two frames. Both frames are arranged so they can be swung simultaneously clear of the bed plate, for dumping.

By means of the crank, shown at the right of Fig. 3, a force (indicated on a spring balance with a capacity of 200 lb.) is applied to the upper frame, causing the material under test to be sheared on a horizontal plane. Since the material is compressed during the test by this horizontal movement, and the upper frame moves several inches before the maximum pull is reached, it is necessary to attach a platform to the end of the lower frame to prevent the upper frame from being emptied prematurely. Means must be provided also, for preventing lifting of the upper frame, or of both frames, when the horizontal pull is applied to the upper frame.

In using this instrument, weights producing pressures up to somewhat over one pound per square inch, including that due to the weight

of the test material in the upper frame, were applied as shown in Fig. 3. It was found in preliminary tests that values of the shear resistance calculated from tests on loosely packed material did not differentiate sufficiently between different mixes. The tests were accordingly made on rodded concretes and on rodded oven-dried aggregate.

Tests made in this manner are sensitive to change in mix but not sensitive to small differences in the amount of mixing water.

d. Stickiness

In the test developed stickiness was measured by the adhesion of the concrete to a plane metal plate. The device is shown schematically in Fig. 4. A steel plate, 11 in. square, is placed on a horizontal surface, a frame 1¾ in. deep placed over it, and the frame filled with concrete

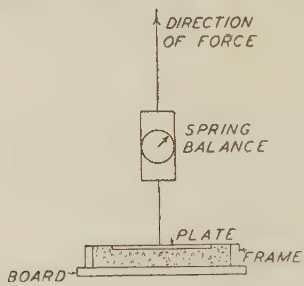

FIG. 4—SCHEMATIC DRAWING OF ADHESION APPARATUS

which is then rodded 75 times uniformly over the area. The plate, frame and concrete are then turned over so that the plate is again horizontal, but now on top of the test concrete. Force is now applied steadily to the center of the plate with increasing magnitude until the plate loosens, and the maximum force applied, which occurs just as the plate loosens, is recorded in pounds per square inch of plate area. Although the plate loosens instantly, concordance of readings is not entirely satisfactory.

4. DISCUSSION OF TEST RESULTS

a. Harshness

A study of concretes with a constant ratio of cement to aggregate (Table 1) shows that the harshness (H) increases from 16 in the over-sanded 1:3:3 mix (No. 1) to 52 in the under-sanded 1:1½:4½ mix (No. 4).

The b-b_o ratio defined by Talbot and Richart(7) is a more satisfactory

[7] University of Illinois Bulletin 137. October, 1923.
b is the absolute volume of the coarse aggregate in a unit volume of concrete.
b_o is the absolute volume of the coarse aggregate in a unit volume of that aggregate (rodded).

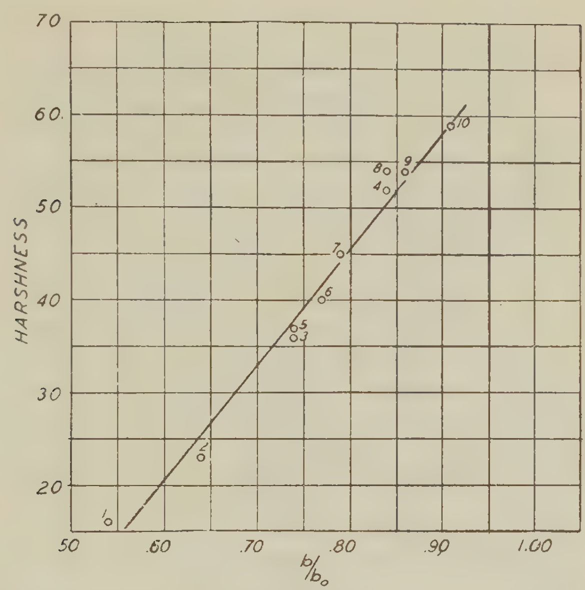

FIG. 5—RELATION BETWEEN HARSHNESS AND B/B°. NUMBERS ACCOMPANYING POINTS INDICATE MIXES IN TABLE 1

value than the sand-gravel ratio for expressing harshness. This ratio has been computed for the mixes given in Table 1 and harshness plotted against it in Fig. 5. There is a distinct trend toward a linear relation between these qualities. Talbot and Richart have stated that the b-b_o ratio is related to the harshness of the mix. The H value gives a definite numerical value of harshness whereas Talbot and Richart estimated harshness by observation.

TABLE 1—HARSHNESS AND SHEAR OF CONCRETES WITH DIFFERENT SAND-GRAVEL RATIOS

Mix No.	Maximum Size of Aggregate Inches	Mix by Weight	Fineness Modulus Including Cement F. M.	b/b_o*	Harshness† H	Resistance to Shear (C) at Flow of 50†, lb/sq. in.
1	¾	1:3:3	4.09	.54	16	1.14
2	¾	1:2½:3½	4.36	.64	23	1.32
3	¾	1:2:4	4.64	.74	36	1.93
4	¾	1:1½:4½	4.91	.84	52	2.06
5	¾	1:2¼:4 2/5	4.60	.74	37	2.00
6	1	1:2:4	5.38	.77	40	2.31
7	1½	1:2:4	5.38	.79	45	3.52
8	1½	1:1 2/5:4 3/5	5.47	.84	54	2.99
9	1½	1:1 1/10:4 9/10	5.69	.86	54	2.88
10	1½	1:1 7/10:6 2/5	5.75	.91	59	3.24

*See footnote 7.
†Cement *A*.

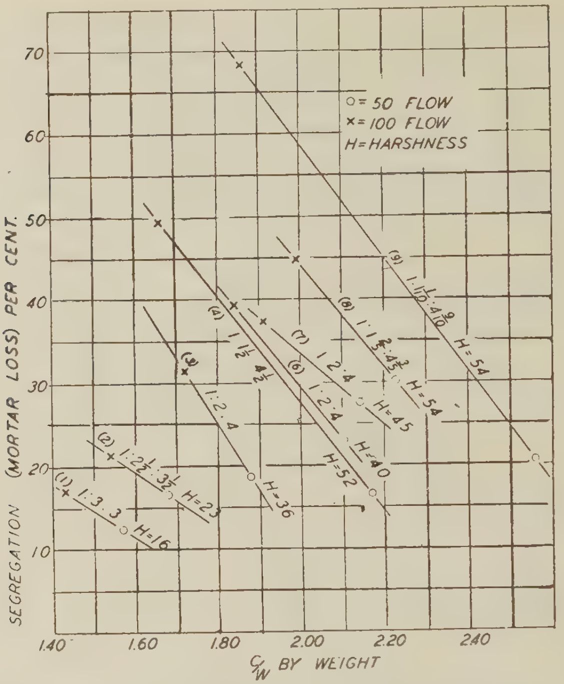

FIG. 6—VARIATION IN SEGREGATION WITH VARIATION IN C/W AND WITH GRADING OF AGGREGATE. NUMBERS IN PARENTHESIS REFER TO THE MIXES IN TABLE 1

Since harshness is determined at only one arbitrarily chosen flow, namely 85, this test cannot be used to measure the effect of different water contents for the same mix.

b. Segregation

Segregation data, as measured by mortar loss, are plotted in Fig. 6 for a group of concretes with the same cement content. It is apparent that there are great differences in the amounts of segregation between the extremes of the over-sanded mix 1:3:3 and the under-sanded mix 1:1 1/10:4 9/10, and also for concretes with the same flow. Differences are accentuated by increasing the flows of concretes from 50 to 100; that is, increase in water necessary to increase flow from 50 to 100 will cause a much larger increase in segregation in under-sanded than in over-sanded concrete.

Since segregation increases with water content and decreases with sand and cement content, the segregation is plotted against ratio of water to cement plus sand in Fig. 7. Values shown in the curve are taken from Tables 2 and 3. It appears that segregation is a function

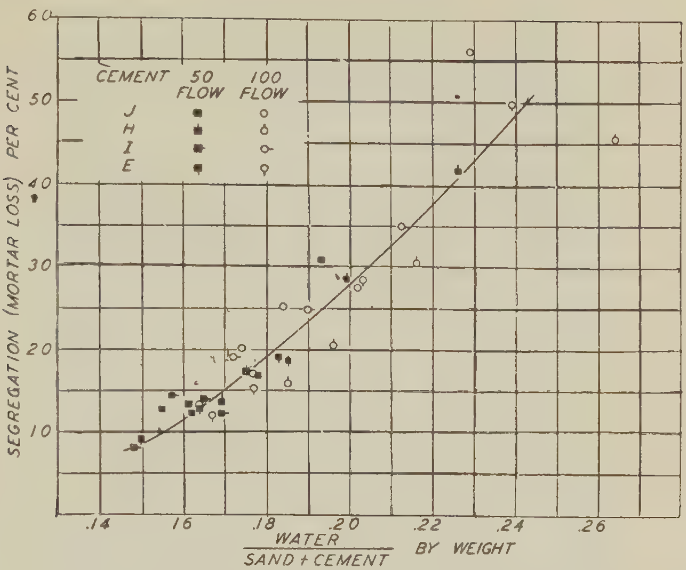

FIG. 7—VARIATION IN AGGREGATION WITH VARIATION IN RATIO OF WATER TO SAND PLUS CEMENT FOR MIXES OF TWO DIFFERENT FLOW VALUES

of this ratio even though the mixes varied in brand of cement, and in water, cement and sand content.

Although an increase in segregation with increase in harshness was noted there was not sufficient correlation between these quantities to forecast satisfactorily the segregation from the harshness.

c. Shear Resistance

Shear tests were made at four pressures varying from that due to the weight of the material above the plane of shear, up to a maximum of slightly over 1 lb. per sq. in. It was found that for any given mix, and for a given flow, the relation between pressure and shear resistance was in agreement with Hatch's(8) equation written in the form $S = CL^m$, as shown by the typical data in Fig. 8. In this equation S is the shearing stress expressed in lb. per sq. in., L is the pressure normal to the plane of shear in lb. per sq. in., and C and m are constants and therefore when $L = 1$ the constant C equals the shearing stress. In making a test the force increased with the travel of the upper frame to a maximum, then suddenly decreased. The maximum shear values were recorded, and the value for unit pressure $L = 1$; obtained by interpolation and expressed as C, is reported in this paper as the measure of shear resistance.

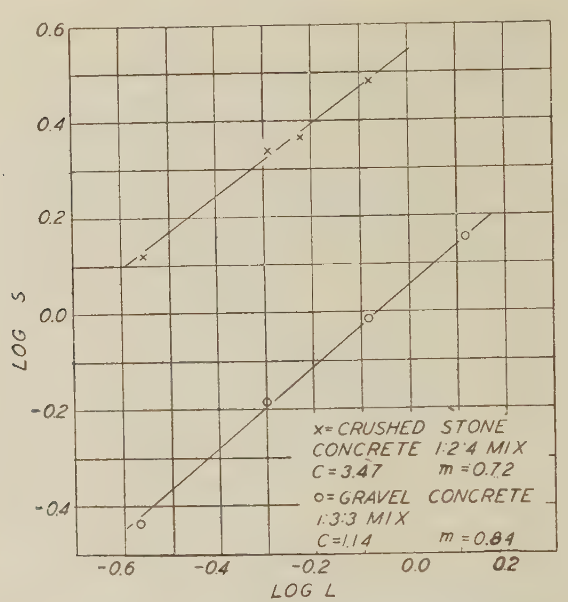

FIG. 8—DIAGRAM SHOWING APPLICATION OF HATCH'S EQUATION ($S = CL^m$)

In Fig. 9 values of C obtained on concrete mixes given in Table 1 are plotted against the fineness modulus. In Fig. 10 values of C obtained with the dry rodded aggregates used in mixes plotted in Fig. 9 are plotted against the fineness modulus of the aggregate alone and, in addition, values are plotted for five aggregates in each of which the particles are approximately all of one size.

Since all intermediate screen sizes of the U. S. Standard Sieve Series are not used in determining fineness modulus, aggregates or concretes of the same modulus might be essentially different in grading and would therefore show a different shear resistance. Thus, of the concretes for which values are shown in Fig. 9, all but Mix 6 and Mix 7 are fairly in accord with the representative line. The shear resistance of 6, (a gap graded mix, with gravel all ¾ to 1 in.) is low, while the resistance of 7 (also a gap graded mix, with gravel all 1 to 1½ in.) is too high. If a larger number of sieves were used in determining fineness modulus the value for Mix 6 would be raised and that for Mix 7 lowered.

As it may be expected that the relations in Fig. 9 and 10 would be modified also if more angular aggregates, or other cements, or a different degree of compacting were used(9), it would seem that C is a

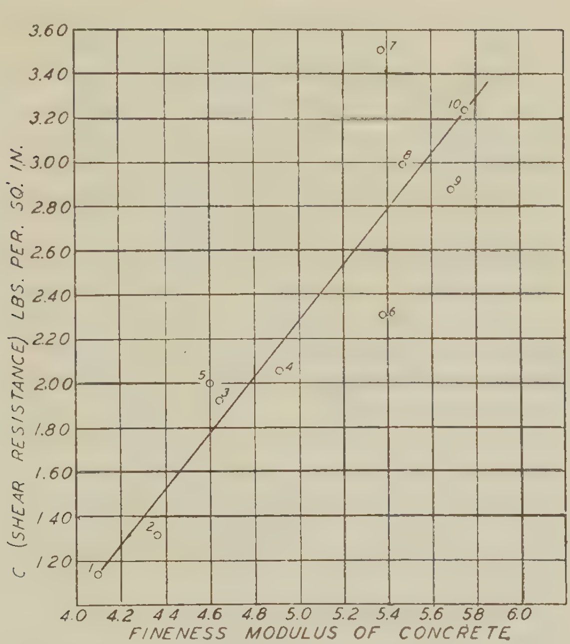

FIG. 9—VARIATION IN C (SHEAR RESISTANCE) WITH VARIATION IN FINENESS MODULUS OF CONCRETE

better indication of the effects of grading than is the fineness modulus, (as calculated from the usual number of sieves) and further should indicate the effect of angularity of the particles of aggregate.

Fig. 11 shows a distinct linear relation between the harshness and shear of the mixes in Table 1. Mixes 7 and 4, which have very unusual grading, are exceptions. It seems, therefore, that the shear test may be substituted for the harshness test (except for patently abnormal mixes) and, since it can be applied over wide ranges of water content, is to be preferred to the harshness test.

It is desired to build ultimately a considerably larger container to enable the testing of concrete with larger aggregate and also to obtain more uniform results.

d. Stickiness

Tables 2 and 3 give the results of adhesion tests of concretes of varying richness.

Since the stickiness of concrete is due solely to the cement paste, it may be expected that adhesion would increase with cement content, and a tendency in this direction is indicated by the limited data of Table 2. Shear resistance, however, decreases with increase in cement

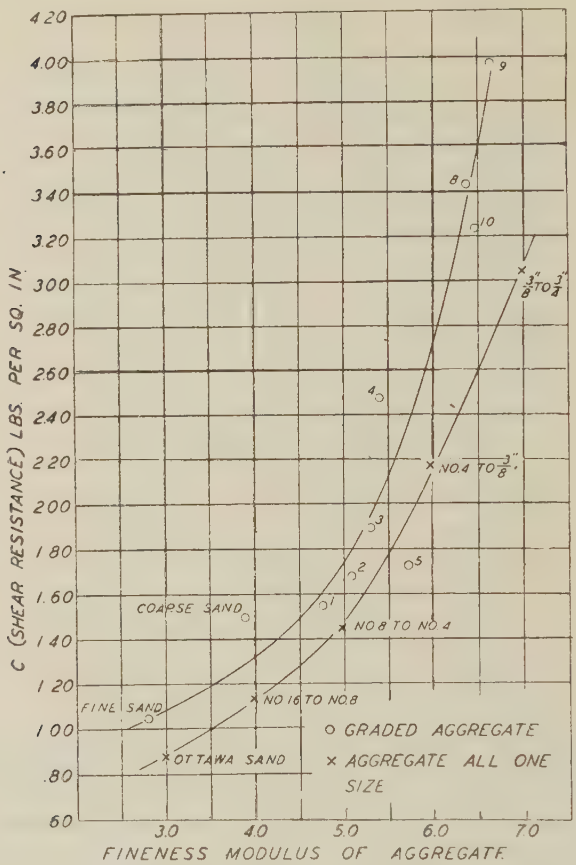

FIG. 10—THE UPPER CURVE SHOWS THE VARIATION IN C (SHEAR RESISTANCE) WITH VARIATION IN FINENESS MODULUS OF TWO SANDS AND OF EIGHT OF THE AGGREGATES OF THE CONCRETES DESCRIBED IN TABLE 1 WHICH CONTAIN GRADED AGGREGATE THE LOWER CURVE SHOWS THE VARIATION IN C (SHEAR RESISTANCE) WITH VARIATION IN FINENESS MODULUS OF FIVE AGGREGATES IN EACH OF WHICH THE PARTICLES ARE APPROXIMATELY OF ONE SIZE

content. Hence in cases where both adhesion and shear resistance are effective, the total resistance might decrease or increase with the cement content, depending on whether adhesion or shear resistance predominated. Purrington and Loring([10]) in tests of the power required to operate a concrete mixer, observed a decrease over a certain range of cement content, and an increase with still richer mixes.

Fig. 12 shows the relation between c/w and adhesion, for a 1:2½:3½ and 1:2:4 mix (No. 2 and 3, Table 1). Points for both mixes lie

TABLE 2—ADHESION AND OTHER TESTS FOR CONCRETES MADE WITH CEMENT J

Mix No.	Mix by Weight	C/W for Flow of		Adhesion for Flow of 50 lb/sq. in.	Segregation (Mortar Loss) for Flow of		Resistance to Shear (*C*) at Flow of 50 lb/sq. in.	Fineness Modulus Including Cement F. M.	Harshness *H*
		50	100		50 %	100 %			
11	1:1½:2	2.50	2.17	.18	13	25	0.51	3.93	30
12	1:2:2 2/3	2.16	1.92	.17	13	20	0.71	4.15	29
13	1:3:4	1.54	1.41	.14	12	17	1.29	4.42	25
14	1:2⅛:5	1.61	1.41	.15	17	28	1.57	4.79	41
15	1:1¼:3¾	2.20	1.94	.15	31	56	1.78	4.79	48

approximately on the same curve, and therefore adhesion of the two mixes would be alike for equal c/w values. Flows of the mixes also given in the figure are, however, markedly different at equal c/w values. Adhesion was practically constant for c/w values of Mix 3 below 1.65 but an increase in c/w values beyond that point markedly increased the adhesion of both mixes. It is interesting to note that the adhesion of the slightly over-sanded mix (1:2½:3½) is less than that of the slightly under-sanded mix (1:2:4) for all points but one (flow = 50), and that the adhesion is lower than that of the under-sanded mix when both have the same flow, irrespective of water content.

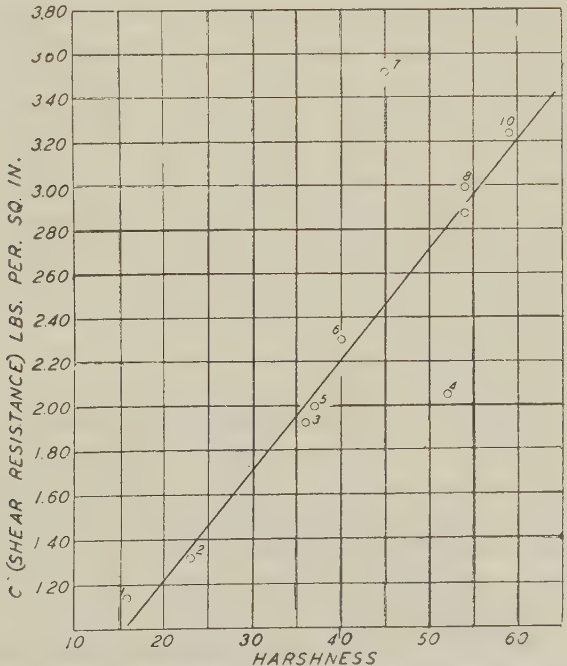

FIG. 11—RELATION BETWEEN SHEAR RESISTANCE AND HARSHNESS. NUMBERS ACCOMPANYING PLOTS INDICATE MIXES IN TABLE 1

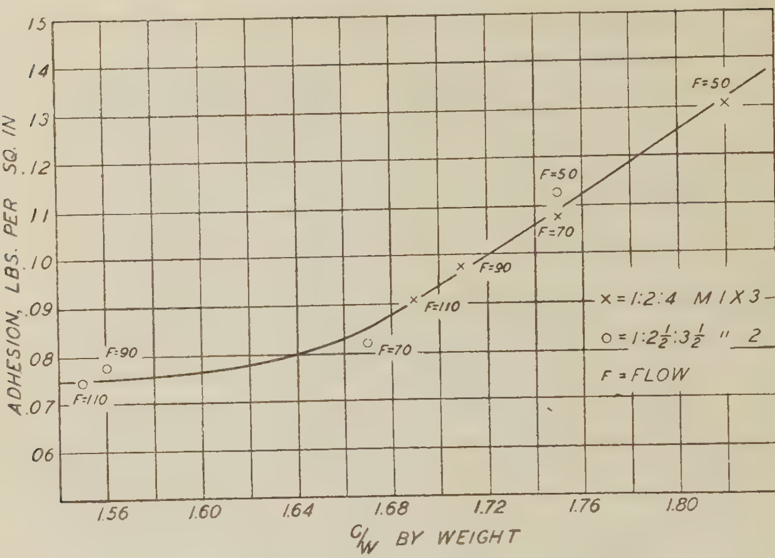

FIG. 12—RELATION BETWEEN ADHESION AND C/W

5. APPLICATION OF THE TESTS TO CONCRETES MADE FROM DIFFERENT CEMENTITIOUS MATERIALS

Table 3 contains certain data obtained in applying the suggested methods for harshness, segregation, shear resistance, and stickiness to 16 concretes in which 3 different cements and one cement with an admixture of diatomaceous silica were used. The concretes differed also from one another in that the sand of the 1:3:3 mixes (a) was reduced by one-half part in each succeeding mix, while the large aggregate was increased by a like amount. One of the outstanding differences in the cements was the fineness to which they were ground. The fineness is given in the table as specific surface in sq. cm. per gram.

Harshness was highest for high early strength cement *H*, which has the greatest specific surface. Variation in harshness of mixes containing this cement, from over-sanded mix *a* to under-sanded mix *d* was, however, only from 36 to 53, whereas variation in harshness in the same mixes containing Cement *I*, with specific surface of 1700 sq. cm. per gram, was from 22 to 51. Harshness based on the average of the 4 mixes of each cement is in inverse order for the 3 cements, *H*, *I* and *E* (specific surfaces of 2100, 1700 and 1440, respectively) while the difference between cements *I* and *E* is very slight. Although the average harshness of the four concretes with diatomaceous silica admixture was lowest of all, it differed but little from the average results

TABLE 3—COMPARISON OF 4 CONCRETES AND 4 CEMENTITIOUS MATERIALS

Mix No.*	Harshness (*H*)	Segregation (Mortar Loss) for Flow of		Resistance to Shear (*C*) at Flow of 50 lb/sq. in.	Adhesion†		C/W for Flow of	
		50 %	100 %		1	2	50	100
					lb/sq. in.			
		CEMENT *H*, Specific Surface, 2100 cm^2/g						
16a	36	14	16	0.62	.15	.12	1.48	1.35
16b	38	17	21	0.80	.19	.15	1.63	1.46
16c	43	29	31	1.06	.22	.16	1.68	1.54
16d	53	42	46	2.09	.21	.13	1.77	1.52
		CEMENT *I*, Specific Surface, 1700 cm^2/g						
17a	22	8	13	1.00	.13	.12	1.69	1.52
17b	24	14	19	1.18	.15	.16	1.82	1.66
17c	40	14	25	1.46	.15	.16	2.02	1.75
17d	51	12	35	2.66	.13	.14	2.37	1.89
		CEMENT *E*, Specific Surface, 1440 cm^2/g						
18a	23	9	12	1.06	.15	.14	1.66	1.50
18b	30	13	15	1.11	.16	.16	1.75	1.61
18c	35	19	29	1.29	.18	.13	1.82	1.64
18d	45	19	50	2.29	.16	.13	2.16	1.68
		CEMENT *E*, With 10% Diatomaceous Silica						
19a	31	7	18	0.85	.17	.18	1.36	1.13
19b	28	12	23	0.74	.17	.17	1.35	1.18
19c	33	18	28	0.88	.19	.18	1.36	1.21
19d	35	24	38	1.50	.16	.15	1.27	1.20

*Mix a = 1:3:3, b = 1:2½:3½, c = 1:2:4, d = 1:1½:4½ — By weight

†Column 1 gives values obtained immediately after mixing; Column 2 gives values obtained after remixing.

obtained with *I* and *E*. This test did not give significantly different values for the effect of an increased fineness of the cement, which usually increases the "fatness." Hence the data indicate that the harshness test could not be used for differentiation of the fineness.

It is evident, however, that harshness is greatly increased as the mixes change from those rich in sand to those lean in sand, except for cement *E* containing diatomaceous silica. In this case there was no practical difference between the under-sanded and over-sanded mix; that is the admixture had reduced the harshness of the under-sanded mix to that of the over-sanded one.

While the percentage mortar loss for the finest cement *H* was decidedly greater when the flow was 50 than for the other cements, the differences were hardly significant when the flow was 100. Concretes made with each cement show an increase in mortar loss with increase in coarse aggregate, when the same flow was maintained. This increase was more pronounced at 100 flow than at 50 flow.

Shear resistance, as may be seen from Table 3, was increased by the use of the cement of intermediate fineness as compared with the use

of the cement having the greatest fineness, while little difference was noted when compared with the coarsest cement. However, shear resistance is sensitive to variations from under- to over-sanding in aggregate grading. The effect of the use of diatomaceous silica is clearly shown by the shear resistance, as may be seen by comparing mixes 18 and 19. It is interesting also to note that the admixture was less effective in the over-sanded than in the under-sanded mixes. The admixture decreased shear resistance of the over-sanded mix from 1.06 to 0.85, a reduction of 20 per cent, whereas the decrease in shear resistance of the under-sanded mix was from 2.29 to 1.50, or 35 per cent.

Adhesion results did not distinguish between cements except for the marked difference in adhesion of concretes made with cements H and E, respectively, before and after remixing. Also, values for this factor are higher generally when an admixture is used. With the other cements, there seemed to be a similarity in adhesion results between over- and under-sanded mixes on the one hand and the two intermediate mixes on the other but, on the whole, the adhesion test does not seem to bring out any significant trend.

6. SUMMARY AND CONCLUSION

1. Four tests were designed to evaluate certain factors of workability which are at least partially independent of one another.

a. Harshness

That relative property of concretes which may be illustrated by the smoothness of an over-sanded, compared with the roughness of an under-sanded, mix.

The harshness test is made by measuring the spread of concretes after 2 drops of the flow table, using mixes having a spread of 85 at 15 drops.

b. Segregation

The partial or complete separation of one class of materials from a uniform mixture of several materials.

The segregation test is made by measuring the amount of mortar separated from concrete by jolting on the flow table.

c. Shear Resistance

The placing of concrete is characterized by the relative movement of innumerable continuous layers of concrete. A fundamental element of this movement is the resistance to shear of a single layer of concrete to an adjacent layer of concrete.

The test for shear resistance is made by measuring the force required to cause shear on a horizontal plane within the concrete, under a normal pressure of one p. s. i.

d. Stickiness

Stickiness is closely related to the adhesion or bond which keeps the concrete as a single mass, in which the individual particles cling together; such a material is distinguished from a pile of dry sand in which the individual particles are kept from moving only by surface friction of the particles.

The stickiness test is made by measuring the vertical force required to separate a horizontal steel plate from the surface of a freshly made concrete.

2. Values obtained by the harshness test bear a straight line relation to the b/b_o ratio proposed by Talbot and Richart. Since the harshness test involves the use of the flow table it has the inherent limitations of the flow table.

3. Potential segregation, as measured by the mortar loss, was found to increase with the flow and with the harshness of the mix. Segregation tests for concretes of equal flow did not measure the effects of addition of admixture or of increased richness in cement.

4. Shear resistance was found, in general, to increase as the fineness modulus of the concrete or of the aggregate increased, but appears to vary also with the grading of aggregates having the same fineness modulus. While it is considered that the test for shear resistance measures essentially the same factor as is measured by the simpler harshness test, the former is to be preferred because it is applicable to any workable concrete.

5. Adhesion was increased slightly by increase in cement content but, for any one mix, decreased with increase in water content.

6. The total number of factors of workability is uncertain, but segregation, stickiness and shear resistance are important.

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For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.*

Sound Absorbing Value of Portland Cement Concrete*

BY F. R. WATSON AND KERON C. MORRICAL†

ABSTRACT

The increasing amount of noise in modern life and the recent movements to minimize unnecessary noise show the trend of public opinion against this nuisance. Also, there is a popular objection to noise in buildings, so that the control of sound is regarded as a necessity to promote the efficiency and comfort of the occupants. Various types of concrete masonry units and acoustical tile have been used for sound absorbing purposes and the Portland Cement Association was led to consider the possibilities of further extending such use and of investigating the sound absorbing values of concretes of different composition and physical properties.

The results show that porous materials absorb sound, but dense materials with small porosity are not efficient in absorbing sound. Haydite and cinder concrete constructions possess effective sound-absorbing values; sand and gravel and limestone concretes were not so good, but most of the specimens are more sound absorbent than the usual hard plasters and cements so that these products can be used advantageously in buildings to reduce noise.

NOISE AND ITS EVIL EFFECTS

If all automobile horns were to be abolished, what would result? There might be some difference of opinion about the increased safety for drivers and pedestrians, but there would be no question about the reduction of noise. Furthermore, would you want to drive your automobile down the street on the rims of the wheels, without tires? Certainly not, it would make a terrible racket, and it would be bad for the auto. But the street cars do this all the time. These noises and others have driven the public to set up organizations in a number of cities to reduce the evil. A similar popular objection is urged against

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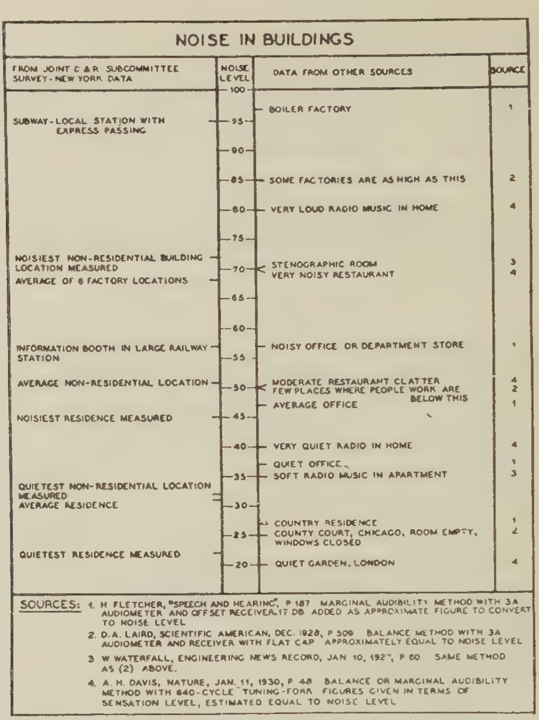

FIG. 1—NOISE LEVEL IN BUILDINGS, EXPRESSED IN DECIBELS

(Tucker, *Journal of the Acoustical Society*, Vol. 2, p. 63, July, 1930)

noise in buildings where typewriters, calculating machines, motors and radios affect the efficiency and comfort of the occupants. Fig. 1 gives a table of the values of noises in buildings.

REDUCTION OF NOISE

One method of reducing noise is to use quieter machines, such as the noiseless typewriter, but more improvement in this regard is needed. Another method of reducing noise is to install materials that absorb sound. Occupants in quiet rooms are less nervous and make fewer mistakes than when they are in noisy rooms. A practical question arises concerning the type of materials efficient in absorbing sound, and this investigation, therefore, included a wide range of concretes, varying from dense, heavy mixes using sand and gravel and limestone aggregates to light, porous ones having varying proportions of cement and light weight aggregates, such as cinders and haydite.

EXPERIMENTAL METHODS

Two methods were used to determine the absorbing value of the different concrete compositions. One method used the Sabine reverberation method in which the time is measured that elapses when various standard sounds die out in a specially prepared room, both with and without the sound absorbing samples. Calculations based on these periods of decay give coefficients of absorption that can be used to determine the area of sound absorbent surfaces made up of these materials that will be needed in any room to control the sound. The time of decay is measured instrumentally using electrical apparatus to generate and to measure sound. The time of decay is determined also with the ear, using a stop watch, thus giving an acceptable comparison with the instrumental results. The latter method was used most extensively in this investigation.

A second method for determining absorbing coefficients requires only a small sample of material, one square foot in area. Sound from an electric loud speaker passes along a tube eight inches in diameter and strikes the absorbing sample, which is mounted at an angle of 45 degrees to the axis of the tube, from which the sound is reflected into a side tube for measurement. Non-absorbers, such as thick glass, will reflect practically 100 per cent of the sound down the side tube, while absorbing materials will reflect amounts of sound that depend on their sound-absorbing efficiencies. This method has the advantage that it gives comparative values of small samples in the development of new materials.

POROSITY AN ESSENTIAL QUALITY OF SOUND ABSORBING MATERIAL

The results obtained (shown in Tables 1 and 2), indicate that *porosity* is the chief property of materials needed for sound absorption. The air sticks to the surface of solids and reduces the back and forth motion of air necessary for the propagation of sound waves. As a result, a frictional action is set up that converts the sound energy into heat. The popular idea that stretched wires or sounding boards reduce sound is fallacious; the wires absorb only a minute fraction of the sound, and the sounding boards simply reflect the waves to make a different arrangement of the sound pattern in the room. Sound energy can be absorbed only by changing it into heat. A porous material is especially efficient in absorbing sound, because the back and forth "pumping" action of the air in the minute pores by the sound waves is strongly resisted by the friction. The heating effect (absorption) depends on the frequency of the sound compared with the diameter and length of the pores. In any case the pores should be small, of the order of a hundredth of an inch, or less, in diameter. Intercommunicating

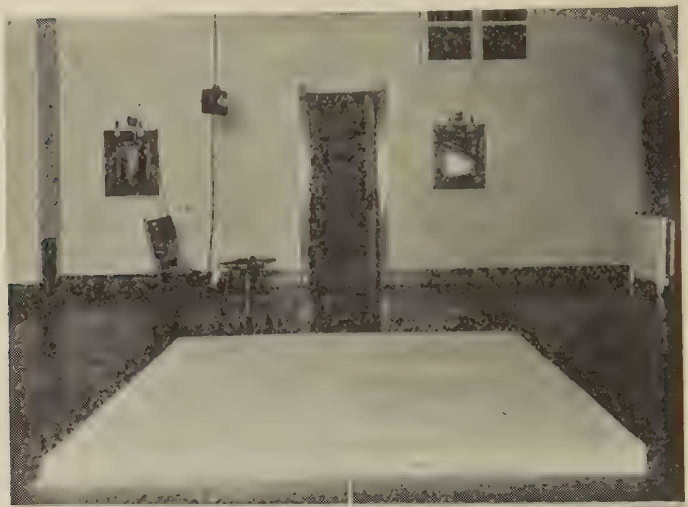

FIG. 2—REVERBERATION ROOM USED FOR MEASURING SOUND ABSORPTION. THE SAMPLE SHOWN ON THE FLOOR IS MADE UP OF STANDARD 4 x 8 x 16 IN., 3-CORE SAND AND GRAVEL CONCRETE PARTITION UNITS

FIG. 3—INSTRUMENT ROOM SHOWING THE ELECTRICAL APPARATUS USED TO GENERATE AND MEASURE SOUND

channels of larger diameter are helpful in allowing free passage of the sound waves into the interior of the material where absorption takes place in the many small pores. It should be remarked that the heating effect due to sound is extremely small. For example, calculations show that a rug, one-fourth inch thick, exposed for 30 minutes to the sound of a vigorous speaker will be raised in temperature only .03° C.

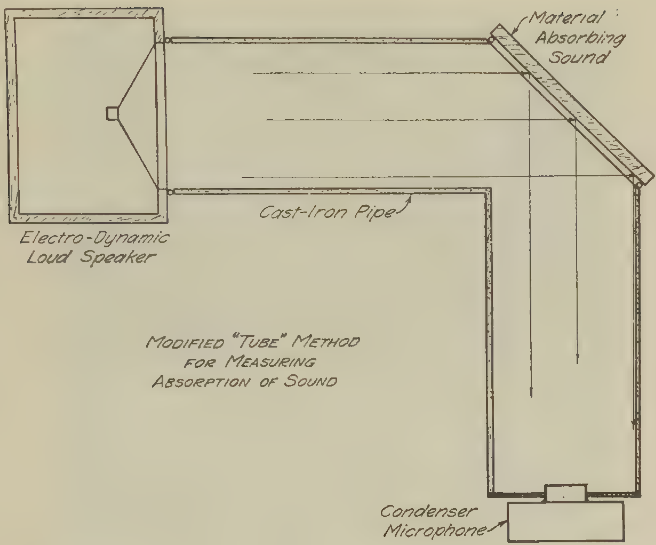

FIG. 4—DIAGRAM OF "TUBE" APPARATUS USED FOR MEASURING SOUND ABSORBING VALUES OF SMALL SAMPLES

This means that the energy of sound is extremely small; a cricket can fill a large church with sound.

EXPERIMENTAL DATA

The advantage of porosity in materials in absorbing sound is shown by the portland cement specimens under investigation. Fig. 5 reproduces photographs of a porous haydite concrete, with a comparatively high sound absorptivity, and of a limestone concrete with little porosity, and low absorptivity. Tables 1 and 2 give numerical data for the sound absorptivity, and Table 3 details of aggregates, mixes and the physical properties of concrete for the specimens tested.

EFFECT OF PAINTING

The effect of paint applied to absorbing materials is readily understood by considering the porosity of the specimen. If the pores are closed by the paint, the absorbing value is taken away. Spray painting has less effect in closing the pores of materials than brush painting. Oil paints reduce sound absorptivity more than water paint. The effect of spray painting with cement and water paint is shown in Table 1, where the average absorptivity was reduced 10 per cent or more.

CONCLUSIONS

The absorption of sound in a room takes place almost entirely at the surfaces where the sound is reflected. The usual interior surfaces—

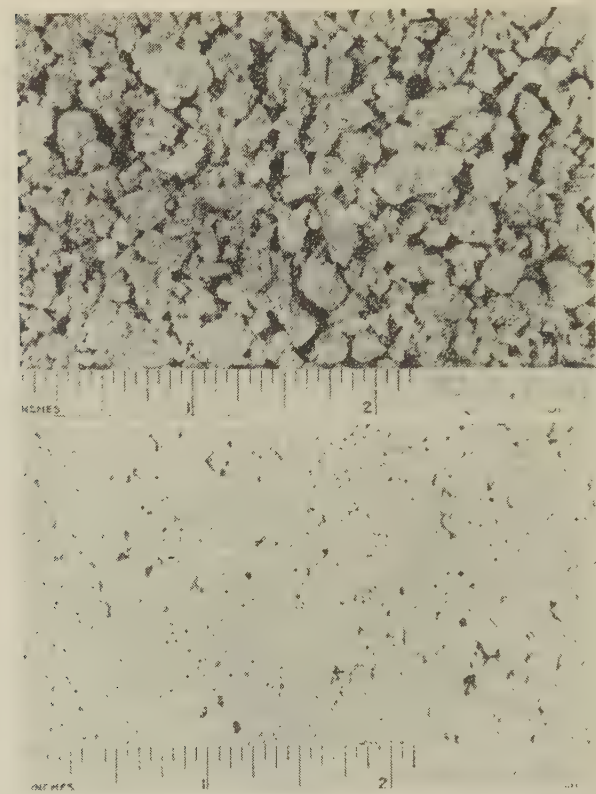

FIG. 5—HAYDITE CONCRETE SPECIMEN WITH A POROUS STRUCTURE AND AN EFFICIENT SOUND ABSORPTIVITY AND (BELOW) A LIMESTONE CONCRETE SPECIMEN WITH LITTLE POROSITY AND A CORRESPONDINGLY SMALL ABSORPTIVITY

(Top) Haydite aggregate; fineness modulus, 4.50; mixture (by vol.) 1:9; absorptivity, 0.54
Limestone aggregate; fineness modulus, 3.50; mixture (by vol.) 1:12; absorptivity, 0.10

hard plaster, wood and glass—absorb only about 3 per cent (absorbing coefficient 0.03) of this incident sound. Any material that absorbs 15 per cent or more is regarded as useful in buildings. Inspection of the absorption coefficients in Tables 1 and 2 show that the haydite and cinder concretes have average coefficients approximating 0.50, meaning that they absorb 50 per cent of the incident sound at each reflection, and that they will be efficient absorbers of sound in rooms. Materials with smaller coefficients, but more than 0.15, are also useful; a building completely lined with such materials would be noticeably quiet compared with present modern buildings where the surfaces absorb about 3 per cent.

As an example consider the quieting effect of using a sound absorbing tile of coefficient 0.18 for the walls and ceiling of a room instead of the inefficient hard plaster of coefficient 0.03. Suppose the room is an

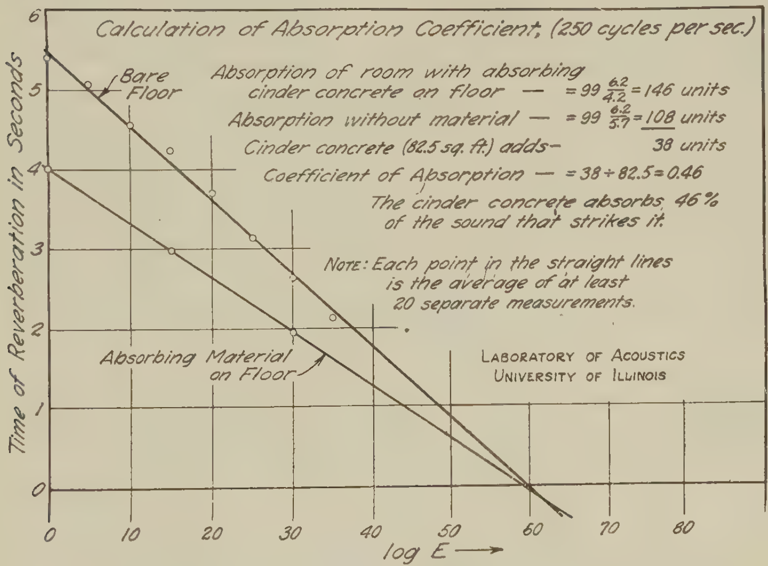

FIG. 6—HOW AN ABSORBING COEFFICIENT IS CALCULATED

office 15 ft. sq. x 10 ft. high, for which the volume is 2250 cu. ft. and that it has a linoleum floor covering. The calculations of the acoustic conditions are as follows:

Hard plaster on walls and ceiling, 825 sq. ft. at 0.03....	24.75 units of absorption
Linoleum on floor, 225 sq. ft. at 0.03...............	6.75 units of absorption
	31.50 units of absorption

The time t taken for an average sound to decay in the room is calculated from the equation: $t = 0.05 \times \text{volume} \div \text{absorption}$, or, $t = 0.05 \times 2250 \div 31.50 = 3.57$ seconds.

With the absorbing tile used instead of the hard plaster, the calculations become:

Absorbing tile, 825 sq. ft. at 0.18..................	148.5 units of absorption
Linoleum, 225 sq. ft. at 0.03......................	6.75 units of absorption
	155.25 units of absorption

$t = 0.05 \times 2250 \div 155.25 = 0.72$ seconds.

These calculations show that the office with hard plaster walls and ceiling would have disturbing sounds that would not disappear for 3.57 seconds, while with the sound absorbing tile, such as the painted sand

and gravel tile (S-2-S in Table 1) the sound would die out in 0.72 seconds, and the room would have the quiet conditions needed to allow the occupants to do efficient work. For larger rooms and auditoriums, it is necessary to use quieting materials with larger coefficients.

TABLE 1—ABSORPTION COEFFICIENTS USING LARGE AREAS OF MATERIALS IN THE REVERBERATION ROOM

Specimens built of 4″ x 8″ x 16″ concrete partition tile

Material	Frequency of Sound					Average Coefficient
	250	500	1000	2000	4000	
	CINDER AGGREGATE CONCRETE					
C-1-S (Unpainted)	0.47	0.51	0.61	0.61	0.55	0.55
C-1-S (Painted)	0.44	0.44	0.52	0.55	0.52	0.49
C-2-S (Unpainted)	0.58	0.44	0.36	0.58	0.58	0.51
C-2-S (Painted)	0.58	0.39	0.33	0.45	0.47	0.44
	SAND AND GRAVEL AGGREGATE CONCRETE					
S-1-S (Unpainted)	0.28	0.35	0.50	0.44	0.36	0.37
S-1-S (Painted)	0.25	0.29	0.28	0.24	0.27	0.27
S-2-S (Unpainted)	0.26	0.35	0.35	0.21	0.16	0.27
S-2-S (Painted)	0.20	0.23	0.18	0.13	0.16	0.18
	LIMESTONE AGGREGATE CONCRETE					
L-1-S	0.21	0.26	0.21	0.19	0.20	0.21
L-2-S	0.23	0.22	0.23	0.23	0.15	0.21
	HAYDITE AGGREGATE CONCRETE					
H-1-S	0.58	0.56	0.33	0.51	0.50	0.50
H-2-S	0.42	0.70	0.33	0.54	0.51	0.50

FIG. 7—TUBE APPARATUS. EXPONENTIAL HORNS WERE USED TO ELIMINATE REFLECTION OF SOUND AT THE OPEN ENDS

The compositions listed in Table 1 consisted of standard 4 x 8 x 16 in., 3-core concrete masonry partition units, made in a power tamper, stripper machine in a

commercial plant. The tile units were laid with ⅜-in. mortar joints so as to give an area of 82.5 sq. ft. The cinder concrete and the sand and gravel concrete tile specimens were tested first plain, then again after spray painting with a portland cement and water paint. Each coefficient recorded is the average of some 150 separate measurements of the time of decay of sound, giving a total of approximately 10,000 measurements for all the coefficients in Table 1. Five standard, pure sounds were used of frequencies 250, 500, 1000, 2000 and 4000 cycles per second, thus giving the coefficients over a wide range. The average of these five values gives a more suitable basis for comparing the different compositions than the values for any one frequency. Inspection of the average values shows that painting reduces the absorbing efficiency, depending on how much the pores are closed to the sound. The cinder and haydite concretes show the greatest sound-absorbing values, while the limestone concretes are the least efficient. All the compositions in Table 1 can be used beneficially for absorbing sound in buildings. Fig. 6 shows the graphs of data and the calculations used in determining an absorption coefficient.

TABLE 2—ABSORPTIVITIES DETERMINED BY THE TUBE METHOD, USING SMALL SPECIMENS

Specimen	Absorptivity		
	Plain	1 Coat Spray Paint	2 Coats Oil Paint
Cinders			
CU 3.50–8	.24		
–6A	.33	.30	
–6B	.26		.08
–9A	.25	.21	
–9B	.24		.10
–12A	.33	.33	
–12B	.25		.18
CU 4.00–6A	.15	.11	
–6B	.14		.06
–9A	.24	.22	
–9B	.25		.12
–12A	.39	.34	
–12B	.43		.20
CU 4.50–6A	.28	.24	
–6B	.29		.07
–9A	.40	.36	
–9B	.34		.18
–12A	.40	.40	
–12B	.34		.25
CJ 3.50–6A	.58	.43	
–6B	.59		.18
–9A	.63	.64	
–9B	.64		.19
–12A	.81	.75	
–12B	.80		.13
CJ 4.00–6A	.53	.50	
–6B	.42		.12
–9A	.64	.62	
–9B	.63		.18
–12A	.68	.59	
–12B	.66		.14
CJ 4.50–6A	.45	.29	
–6B	.49		.19
–9A	.47	.43	
–9B	.55		.28
–12A	.64	.60	
–12B	.65		.21
2" CU 3.50–9	.40		
–12	.38		
4.50–9	.28		
–12	.33		

Specimen	Absorptivity	
	Plain	1 Coat Spray Paint
Haydite		
HU 3.50–6	.34	
–9	.18	
–12	.27	
HU 4.00–6	.35	
–9	.34	
–12	.55	
HU 4.50–6	.51	
–9	.54	
–12	.52	
HJ 3.50–6	.68	
–9	.75	
–12	.79	
HJ 4.00–6	.64	
–9	.68	
–12	.71	
HJ 4.50–6	.54	
–9	.61	
–12	.72	
Sand and Gravel		
SGU 3.50–8	.21	.16
–12	.20	.06
4.00–8	.29	.24
–12	.20	.13
4.50–8	.32	.18
–12	.27	.15
SJ 3.50–8	.27	.24
–12	.41	.39
4.00–8	.19	.17
–12	.40	.27
4.50–8	.09	.08
–12	.27	.19
Limestone		
LU 3.50–8	.10	
–12	.10	
4.50–8	.10	
–12	.10	
LJ 3.50–8	.58	
–12	.60	
4.50–8	.57	
–12	.62	

Results of Successive Coats of Paint

	Plain	1	2	3	4	
CU 4.00–9A	.24	.22	.19	.09	.10	portland cement spray paint
CU 4.00–9B	.25	.20	.12	.08	.05	oil paint

The specimens listed in Table 2 were all 14 x 14 in. in area, with a thickness of either 1 in. or 2 in. These specimens were made of a normal block mix consistency concrete tamped in flat molds on a wood pallet. The bottom face was exposed to the incident sound, in testing. The 1:12 mix specimens of the "J" group were friable due to the lack of fines in the mix. The frequency used was 500 cycles per second, and each absorbing value recorded is the average of at least 4 separate determinations, each determination involving 6 measurements. The absorbing values give a basis of comparison of different specimens. It is to be noted that the specimens made of light weight aggregates, such as haydite and cinders, with large porosity, have more efficient absorption than the denser limestone concrete specimens with little porosity. See Fig. 5. The results of the "J" group suggest the advantage of reducing the amount of fines in the aggregate. The sand and gravel concrete specimens were fairly good absorbers. In general the lean mix specimens gave higher values than those made of richer mixes. The grading of the aggregate appeared to be a factor which, however, was not consistent for all types of aggregates. The limestone specimens, LU and LJ, exhibit very different absorptivities. This is probably due to the different particle size distribution of the aggregate and to differences in the surface finish, the LJ specimens having a rough texture so that the pores were exposed, thus allowing a considerable absorption of sound, while the surfaces of the LU specimens were smoother with the pores closed to the sound. Painting the specimens reduced the absorptivity in proportion to the closing of the pores by the paint. Fig. 7 shows the tube apparatus.

This paper is concluded with Table 3 on the 3 following pages:

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.*

TABLE 3—DETAILS OF AGGREGATES, MIXES AND PHYSICAL PROPERTIES OF CONCRETE IN UNITS FOR SERIES A AND B SOUND ABSORPTION TESTS

Specimen Identification Number	AGGREGATE										CONCRETE				
	Type	Grading—Per Cent Retained on Screen							Fineness Modulus	Dry Rodded Weight Lb. per Cu. ft.	Cu. Ft. Dry Rodded Aggregate Used Per Sack Cement	Weight of Dry Concrete Lb. per cu. ft.	Compressive Strength Lb. per sq. in. Gross Area	Water Absorption	
		3/8"	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100						% of Dry Weight of Concrete	%l By Voume of Concrete
Series "A" (4" x 8" x 16" Hollow Concrete Tile)															
C-1-S	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	8	83.2	460	16.2	21.5
C-2-S	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	8	82.2	560	14.1	18.6
H-1-S	Haydite	0	20.4	39.8	59.1	70.7	77.6	82.7	3.50	54.0	9	67.5	880	20.5	22.2
H-2-S	Haydite	0	41.6	68.2	78.3	84.2	87.6	90.1	4.50	49.6	9	59.5	610	15.3	14.5
S-1-S	Sd. & Gr.	0	6.96	26.7	49.9	72.7	95.2	99.4	3.50	113.5	10	117.4	980	9.1	17.1
S-2-S	Sd. & Gr.	0	38.0	59.0	72.2	84.7	97.0	99.2	4.50	117.6	10	127.5	1470	5.9	12.1
L-1-S	Limestone	0	13.6	40.0	58.0	72.1	80.7	86.3	3.50	111.0	10	123.1	1180	8.6	16.9
L-2-S	Limestone	0	46.1	64.6	75.2	83.3	88.2	91.4	4.50	104.75	10	123.0	1280	7.7	15.2
Series "B" (14" x 14" x 1" and 2" Thick Slabs)*—Normal Grading															
CU-3.50-8	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	8	—	No Tests Made	17.9	—
CU-3.50-6	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	6	—		16.7	—
CU-3.50-9	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	9	—		19.1	—
CU-3.50-12	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	12	—		20.5	—
CU-4.00-6	Cinders	0	22.8	49.8	68.6	80.4	87.0	91.2	4.00	66.0	6	—		14.6	—
CU-4.00-9	Cinders	0	22.8	49.8	68.6	80.4	87.0	91.2	4.00	66.0	9	—		15.6	—
CU-4.00-12	Cinders	0	22.8	49.8	68.6	80.4	87.0	91.2	4.00	66.0	12	—		18.2	—
CU-4.50 6	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	6	—		14.1	—
CU-4.50-9	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	9	—		15.7	—
CU-4.50-12	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	12	—		17.0	—
CU-3.50-9*	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	9	—		19.3	—
CU-3.50-12*	Cinders	0	4.5	36.0	60.8	76.4	84.7	89.7	3.50	67.0	12	—		21.1	—
CU-4.50-9*	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	9	—		16.5	—
CU-4.50-12*	Cinders	0	42.2	64.3	76.7	84.5	89.3	92.7	4.50	62.8	12	—		17.4	—
HU-3.50-6	Haydite	0	20.4	39.8	59.1	70.7	77.6	82.7	3.50	54.0	6	—		19.9	—

*2" Thick slabs indicated thus, all others are 1" thick.

(Table 3 continued next page)

TABLE 3 (continued)—DETAILS OF AGGREGATES, MIXES AND PHYSICAL PROPERTIES OF CONCRETES IN UNITS FOR SERIES A AND B SOUND ABSORPTION TESTS (Continued)

Specimen Identification Number	AGGREGATE: Type	Grading—Per Cent Retained on Screen: 3/8"	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100	Fineness Modulus	CONCRETE: Dry Rodded Weight Lb. per Cu. ft.	Cu. Ft. Dry Rodded Aggregate Used Per Sack Cement	Weight of Dry Concrete Lb. per cu. ft.	Compressive Strength Lb. per sq. in. Gross Area	Water Absorption: % of Dry Weight of Concrete	% By Volume of Concrete
Series "B" (14" x 14" x 1" and 2" Thick Slabs)*—Normal Grading (Cont'd.)															
HU-3.50-9	Haydite	0	20.4	39.8	59.1	70.7	77.6	82.7	3.50	54.0	9	—		22.8	—
HU-3.50-12	Haydite	0	20.4	39.8	59.1	70.7	77.6	82.7	3.50	54.0	12	—		25.8	—
HU-4.00-6	Haydite	0	31.0	54.0	68.7	77.5	82.6	86.6	4.00	51.25	6	—		21.2	—
HU-4.00-9	Haydite	0	31.0	54.0	68.7	77.5	82.6	86.6	4.00	51.25	9	—		21.1	—
HU-4.00-12	Haydite	0	31.0	54.0	68.7	77.5	82.6	86.6	4.00	51.25	12	—	N	20.8	—
HU-4.50-6	Haydite	0	41.6	68.2	78.3	84.2	87.6	90.1	4.50	49.6	6	—	o	15.8	—
HU-4.50-9	Haydite	0	41.6	68.2	78.3	84.2	87.6	90.1	4.50	49.6	9	—	T	17.5	—
HU-4.50-12	Haydite	0	41.6	68.2	78.3	84.2	87.6	90.1	4.50	49.6	12	—	e	19.9	—
SU-3.50-8	Sd. & Gr.	0	6.96	26.7	49.9	72.7	95.2	99.4	3.50	113.5	8	—	s	8.6	—
SU-3.50-12	Sd. & Gr.	0	6.96	26.7	49.9	72.7	95.2	99.4	3.50	113.5	12	—	t	8.8	—
SU-4.00-8	Sd. & Gr.	0	22.4	42.8	61.0	78.6	96.0	99.3	4.00	118.25	8	—	s	8.6	—
SU-4.00-12	Sd. & Gr.	0	22.4	42.8	61.0	78.6	96.0	99.3	4.00	118.25	12	—	M	8.6	—
SU-4.50-8	Sd. & Gr.	0	38.0	59.0	72.2	84.7	97.0	99.2	4.50	117.60	8	—	a	7.9	—
SU-4.50-12	Sd. & Gr.	0	38.0	59.0	72.2	84.7	97.0	99.2	4.50	117.60	12	—	d	7.5	—
LU-3.50-8	Limestone	0	13.6	40.0	58.0	72.1	80.7	86.3	3.50	111.0	8	—	e	8.3	—
LU-3.50-12	Limestone	0	13.6	40.0	58.0	72.1	80.7	86.3	3.50	111.0	12	—		8.5	—
LU-4.50-8	Limestone	0	46.9	64.6	75.2	83.3	88.2	91.4	4.50	104.75	8	—		7.7	—
LU-4.50-12	Limestone	0	46.9	64.6	75.2	83.3	88.2	91.4	4.50	104.75	12	—		7.5	—
Series "B" (14" x 14" x 1" Thick Slabs)—Step Grading															
CJ-3.50-6	Cinders	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	59.8	6	—		20.2	—
CJ-3.50-9	Cinders	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	59.8	9	—		29.1	—
CJ-3.50-12	Cinders	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	59.8	12	—		30.2	—
CJ-4.00-6	Cinders	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	62.9	6	—		17.9	—
CJ-4.00-9	Cinders	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	62.9	9	—		20.2	—
CJ-4.00-12	Cinders	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	62.9	12	—		21.0	—

*2" Thick slabs indicated thus, all others are 1" thick.

(Table 3 concluded next page)

TABLE 3 (concluded)—DETAILS OF AGGREGATES, MIXES AND PHYSICAL PROPERTIES OF CONCRETE IN UNITS FOR SERIES A AND B SOUND ABSORPTION TESTS (Continued)

Specimen Identification Number	AGGREGATE Type	Grading—Per Cent Retained on Screen 3/8"	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100	Fineness Modulus	CONCRETE Dry Rodded Weight Lb. per Cu. ft.	Cu. Ft. Dry Rodded Aggregate Used Per Sack Cement	Weight of Dry Concrete —b. per cu. ft.	Compressive Strength Lb. per sq. in. Gross Area	Water Absorption % of Dry Weight of Concrete	% By Volume of Concrete
Series "B" (14" x 14" x 1" Thick Slabs)—Step Grading (Cont'd.)															
CJ-4.50-6	Cinders	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	65.7	6	—		15.6	—
CJ-4.50-9	Cinders	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	65.7	9	—		17.9	—
CJ-4.50-12	Cinders	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	65.7	12	—		21.3	—
HJ-3.50-6	Haydite	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	46.1	6	—		31.6	—
HJ-3.50-9	Haydite	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	46.1	9	—		34.6	—
HJ-3.50-12	Haydite	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	46.1	12	—		38.3	—
HJ-4.00-6	Haydite	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	48.2	6	—	No Tests Made	27.3	—
HJ-4.00-9	Haydite	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	48.2	9	—		29.3	—
HJ-4.00-12	Haydite	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	48.2	12	—		32.6	—
HJ-4.50-6	Haydite	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	47.8	6	—		25.0	—
HJ-4.50-9	Haydite	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	47.8	9	—		26.0	—
HJ-4.50-12	Haydite	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	47.8	12	—		30.0	—
SJ-3.50-8	Sd. & Gr.	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	110.9	8	—		9.3	—
SJ-3.50-12	Sd. & Gr.	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	110.9	12	—		10.4	—
SJ-4.00-8	Sd. & Gr.	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	116.3	8	—		7.9	—
SJ-4.00-12	Sd. & Gr.	0	16.0	34.0	50.0	100.0	100.0	100.0	4.00	116.3	12	—		9.4	—
SJ-4.50-8	Sd. & Gr.	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	120.0	8	—		7.0	—
SJ-4.50-12	Sd. & Gr.	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	120.0	12	—		8.1	—
LJ-3.50-8	Limestone	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	98.2	8	—		13.1	—
LJ-3.50-12	Limestone	0	8.0	16.0	26.0	100.0	100.0	100.0	3.50	98.2	12	—		14.8	—
LJ-4.50-8	Limestone	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	105.5	8	—		10.8	—
LJ-4.50-12	Limestone	0	25.0	50.0	75.0	100.0	100.0	100.0	4.50	105.5	12	—		12.7	—

*2" Thick slabs indicated thus, all others are 1" thick.

High Early Strength Cements in Concrete Masonry Manufacture*

Report of Committee 710, Use of High Early Strength Cements in Concrete Products†

BENJAMIN WILK, CHAIRMAN

INTRODUCTION

Committee 710 was organized in 1935 to study the use of high early strengths cements in concrete products manufacture—an outgrowth of interest in the paper by Benjamin Wilk, at the 1935 Convention.[1]

This is a progress report and covers laboratory and plant tests on effect of grading of aggregate, kind of aggregate, volume change, and resistance to freezing and thawing as influenced by the type of cement.

The tests were made at the laboratories of the Lehigh Portland Cement Co., Allentown, Pa., at Lehigh University, Bethlehem, Pa., and at the plant of the Standard Building Products Co., Detroit, Mich.

TESTS AT LEHIGH UNIVERSITY

Since, in the early tests Wilk found that 70-lb. of high early strength cement produced concrete masonry units comparable in strength to that of units made with 94-lb. of normal cement, it was decided in this investigation to make all tests and comparisons on this basis. To make the tests more comprehensive, two normal and two high early strength cements were used.

The concrete mixes were limited to the semi-dry mixes generally used in concrete products manufacture. The consistency of the concrete was such that under heavy tamping indications of water marks would occur on the walls of the units. Three types of aggregates were used: sand and gravel, crushed air-cooled slag, and cinders. The major part of the tests was carried out on 3 by 6-in. concrete cylinders, while for compression and freezing and thawing tests 8 by 8 by 8-in.

*Presented at the 32nd Annual Convention, American Concrete Institute, Chicago, Feb. 25-27, 1936.
†Members of Committee 710: Benjamin Wilk, Chairman, J. C. Pearson, Inge Lyse, Austin Crabbs, A. G. Timms, P. M. Woodworth

[1]"High Early Strength Cement in Concrete Products Manufacture," Journal, Am. Concrete Inst., Jan.-Feb., 1935, *Proceedings*, Vol. 31, p. 241.

block were also used for comparison. For the cylinder tests three specimens of a kind were used for each age, while for the block the number was either two or three. All specimens were cured in the moist room for three days and the remaining time in the air of the laboratory.

The three test ages were 4, 7 and 28 days for the compressive strength specimens. Observations on the volume change specimens were begun immediately upon their removal from the moist room. The freezing and thawing tests of the blocks were begun at the age of 28 days and of the 3 by 6-in. cylinders at 4, 7 and 28 days.

One mix and one consistency were used in that part of the investigation which has been completed to date. Additional tests are planned for concretes of various consistencies and cement contents.

The cements came from two plants located in the Lehigh Valley. Each plant submitted one sample of normal portland and one of high early strength cement. The sand and gravel came from the Morrisville plant of Warner Brothers, Philadelphia. The slag was regular air-cooled blast furnace slag from the Bethlehem Steel Co. and the cinders came from the Ormrod plant of the Lehigh Portland Cement Co. The combined aggregates had a fineness modulus of approximately 3.8.

For the sand and gravel aggregate the concrete mix was 1:5.1:1.72 with a gross cement-water ratio of 1.82 for the normal portland cements, and 1:6.83:2.30 with a cement-water ratio of 1.36 for the high early strength cements.

For the crushed slag the proportions were the same as for sand and gravel aggregates, but the cement-water ratios were 1.44 and 1.08 respectively for the normal portland and the high early strength cements.

For the cinders the proportions were 1:2.90:0.97 and 1:3.87:1.29, with cement-water ratios of 0.88 and 0.66 respectively for the two types of cement.

The cement content of the concrete blocks was found to be 4.65 lb. per block for normal portland cement and 3.50 lb. per block for high early strength cement.

RESULTS OF LEHIGH UNIVERSITY TESTS

Compression Tests

The average compressive strengths of the 3 by 6-in. cylinders are shown in Table 1 and the average block strengths in Table 2. In general, the high early strength cement produced average strengths above those obtained for normal portland cements at all ages of test and for all aggregates. However, there were many cases where in-

dividual strengths of cylinders made with normal portland cement gave higher strengths than the lower of the two high early strength cements.

TABLE 1—COMPARATIVE STRENGTHS OF HIGH EARLY STRENGTH AND NORMAL PORTLAND CEMENTS

Test specimens 3 by 6-in. cylinders.
Specimens made at 70°F. and cured 3 days moist at 70°F., followed by storage in air of laboratory until time for tests.
Fineness modulus of aggregate 3.8.
70-lb. of high early strength cement per batch compared with 94-lb. normal portland cement per batch.
Specimens made and tested at Lehigh University.

Cement	Compressive Strength—p. s. i.								
	Cinder Concrete			Gravel Concrete			Slag Concrete		
	4D.	7D.	28D.	4D.	7D.	28D.	5D.	7D.	28D.
Normal Portland									
N1	870	1090	2200	1760	2490	3150	2220	2320	3460
N2	980	1390	2290	1820	1680*	3350	2210	2570	4240
Average	925	1240	2245	1790	2490	3250	2215	2445	3850
High Early Strength									
H1	1200	1440	2150	2270	3060	3600	2740	2740	4910
H2	1200	1360	2280	2330	2890	3640	2450	2540	3870
Average	1200	1400	2215	2300	2975	3620	2595	2640	4390
Percentage Increase	30	13	—1	28	15	11	17	8	14

*Omitted from average.

The average per cent increase was highest at 4 days. At 28 days the average increase in strength due to the use of high early strength cement was 8 per cent.

The volume change observations and the freezing and thawing tests have been carried on for some time, but the results to date have not progressed far enough to show any major differences in performance for the two types of units.

TABLE 2—AVERAGE STRENGTH OF 8 BY 8-IN. HOLLOW BLOCKS MADE WITH GRAVEL AGGREGATE

Cement	Compressive Strength—p. s. i.		
	4D.	7D.	28D.
N1	855*	1130*	1530*
N2	515	575	1130
H1	680	940	1330
H2	605	905	1405

*The blocks for cement N1 were made first and were tamped more vigorously than the blocks made with the other cements.

TESTS AT CENTRAL LABORATORY, LEHIGH PORTLAND CEMENT CO.

That portion of the committee's work referring to the effect of the grading of aggregates was carried on at the Central Laboratory of the Lehigh Portland Cement Co. The gravel, slag, and cinders were the same as those used in the tests at Lehigh University, but considerable

work was done in separating and recombining the aggregates to definite gradations. The cements used were the brands designated as N1 (normal portland) and H1 (high early strength) in the Lehigh University tests but were taken from different lots.

In general, the same procedure was followed as at Lehigh University; the batches used were designed to give a yield equivalent to one standard building unit (nominal size 8 by 8 by 16 in., 50 per cent core space) for each 3.56-lb. of high early strength cement (75 per cent of the quantity of normal portland). The water content was based on judgment, and in general was such that water would appear at the surface when the 3 by 6-in. test cylinders, well tamped, were nearly filled.

The gradings were selected to show the effect of material finer than the No. 50 sieve, and were designated ⅜ in. to No. 50, ⅜ in. to No. 100, and ⅜ in. to No. 200, although the last mentioned contained in all cases some material finer than No. 200. The program involved the following variables:

2 cements: normal portland and high early strength portland.
3 aggregates: gravel, slag, and cinders.
3 gradings: as indicated in Table 3.
3 ages at test: 4, 7, and 28 days.
1 curing: 3 days in damp room, then in laboratory air.
3 companion specimens at each age, tested in room-dry condition.

TABLE 3—GRADINGS OF AGGREGATES

Sieve	Per Cent Passing Sieve Sizes Indicated					
	Gravel			Slag and Cinders		
	⅜—50	⅜—100	⅜—200	⅜—50	⅜—100	⅜—200
⅜ in.	100	100	100	100	100	100
No. 4	76	77	77	80	81	81
8	54	57	58	60	63	64
16	34	40	42	40	46	49
30	16	24	28	20	30	35
50	0	11	17	0	15	22
100		0	8		0	10
200			3			5*
F. M.	4.20	3.91	3.70	4.00	3.65	3.39

*Slag, 5% finer than No. 200; cinders, 6.5% finer than No. 200.

Fabrication of the cylinders was so conducted as to minimize the effect of variations in hand tamping. Thus a 9-cylinder batch was made for each aggregate grading, and 3 operators each made 3 cylinders from each batch. The 3 companion cylinders for each age were then selected by taking a cylinder made by each operator. The cement content was determined from the net weight of the 9 cylinders of a batch, comprising .2194 cu. ft. of concrete. In some cases the batches were not sufficient to fill the ninth cylinder, and in such cases the net

weight of 8 cylinders, i.e., of .195 cu. ft. of concrete, was used.

As the cement content of the batches varied more or less from the design figures of 4.75 lb. per block for the normal portland cement, and 3.56 lb. per block for the high early strength cement, respectively, Table 4 has been prepared to make the values comparable on the designed figures. The per cent increase of the high early strength cement strengths over the normal portland cement strengths are also shown.

TABLE 4—EFFECT OF GRADING ON COMPRESSIVE STRENGTH

Age Days	⅜—50 Agg. Cement Normal	⅜—50 Agg. Cement H.E.S.	⅜—50 Agg. % Increase	⅜—100 Agg. Cement Normal	⅜—100 Agg. Cement H.E.S.	⅜—100 Agg. % Increase	⅜—200 Agg. Cement Normal	⅜—200 Agg. Cement H.E.S.	⅜—200 Agg. % Increase
				Gravel					
4	2020	2565	27	2140	2420	13	2210	2860	29
7	2560	3375	32	2650	3250	23	3050	3570	17
28	2785	3780	36	3030	3590	18	3310	4110	.24
				Slag					
4	2560	2875	12	2235	2840	27	2130	2500	17
7	3030	3320	9	2780	3560	28	2840	3250	14
28	4385	4460	2	4035	4610	14	3930	4100	4
				Cinders					
4	950	1060	12	995	1165	17	1090	1160	6
7	1360	1410	4	1345	1550	15	1585	1560	—2
28	1590	1730	9	1860	1930	4	1895	1950	3

Table 4 indicates the effects of aggregate grading upon concrete made from the two types of cement. In the case of the gravel aggregate, it is seen that the strengths for both types of cement tend to be higher with the finer aggregate grading, although there is a slight departure from this trend in the high early strength mix with the medium grading. This slight falling off may be accounted for by excess water, as this particular series was noted as being over-wet as judged by early appearance of water in tamping the cylinders. On the other hand, the per cent increase of the high early strength cement concrete over the normal portland is highest in the coarsest grading, suggesting that while the finer aggregate grading benefits both cements, it is somewhat more beneficial to the normal than to the high early strength. The average increase in compressive strength for all ages and grading with this aggregate was 24 per cent.

With the slag aggregate, highest strengths are shown for the normal portland cement in the coarsest grading, and highest for the high early strength cement are shown in the medium grading. The per cent in-

crease of high early strength cement concrete strengths over normal are also highest in the medium mix. The average increase in strength was 14 per cent with this aggregate.

With the cinder aggregates, the strengths are generally highest in the finest grading, although the high early strength cement concrete strengths are substantially the same in the medium and finest gradings. Here again the ratios of high early strength to normal portland cement concrete strengths are a maximum in the medium grading. The increase in strength due to high early strength cement was only 7 per cent when cinders were used as aggregate.

On the whole, the indications from these tests are that gradings of the usual type best suited for lean dry-tamp mixtures should be carried below the 100-mesh size, regardless of the type of cement used, but the superiority of the finer gradings is not marked.

TESTS AT STANDARD BUILDING PRODUCTS CO. PLANT, DETROIT

To check the results reported to the 1935 convention in the paper by Mr. Wilk and to show the relationship between laboratory-made specimens and plant-made specimens, several series of tests were made using the methods reported in the paper.

Table 5 shows the comparative strengths of block made from batches containing 70-lb. of high early strength cement or 94-lb. of normal portland cement.

To determine the effect of using less high early strength cement per batch, additional tests were made, the results of which are shown in Tables 6 and 7.

TABLE 5—COMPARATIVE STRENGTHS OF HIGH EARLY STRENGTH AND NORMAL PORTLAND CEMENTS—JULY SERIES

Test specimens 8 by 8 by 16-in. hollow concrete block.
Net bearing area 50% of gross area.
Specimens made at 70°F., cured 1 day at 90°F., followed by outdoor air storage.
70-lb. of high early strength cement per batch, compared with 94-lb. normal portland cement per batch.
Each value is the average of tests of three specimens.

Cement	Compressive Strength—p. s. i.			
	2D.	4D.	7D.	28D.
Normal Portland				
A	439	531	530	872
B	455	599	679	889
Average	447	565	605	880
High Early Strength				
C	565	660	764	941
D	492	635	745	928
Average	528	647	754	935
Percentage Increase	18	15	25	6

TABLE 6—COMPARATIVE STRENGTHS OF HIGH EARLY STRENGTH AND NORMAL PORTLAND CEMENTS—OCTOBER SERIES

Test specimens 8 by 8 by 16-in. hollow concrete block.

Net bearing area 50% of gross area.

Specimens made at 50°F., cured one day at 90°F. followed by outdoor storage.

62½-lb. of high early strength cement per batch, compared with 94-lb. normal portland cement per batch.

Each value is the average of tests of three specimens.

Cement	Compressive Strength—p. s. i. Gross Area		
	3D.	7D.	28D.
Normal Portland B	565	768	930
High Early Strength C	688	910	949
E	626	857	916
Average	657	884	933
Percentage Increase	16	15	0

TABLE 7—COMPARATIVE STRENGTHS OF HIGH EARLY STRENGTH AND NORMAL PORTLAND CEMENTS—DECEMBER SERIES

Test specimens 8 by 8 by 16-in. hollow concrete block.

Net bearing area 50% of gross area.

Specimens made at 50°F., cured one day at 70°F. followed by outdoor air storage below 32°F.

62½-lb. of high early strength cement per batch, compared with 94-lb. normal portland cement per batch. Each value is the average of tests of three specimens.

Cement	Compressive Strength—p. s. i. Gross Area		
	4D.	7D.	28D.
Normal Portland B	269	504	740
High Early Strength C	208	522	726
D	264	511	741
Average	236	517	734
Percentage Increase	—12	3	—1

STORAGE OF HIGH EARLY STRENGTH CEMENT

During the last year, a large amount of high early strength cement has been used in the plant of the Standard Building Products Company and a number of brands have been tried.

Experience has shown that storage conditions must be carefully watched, especially if the cement is handled in cloth bags. The extreme fineness of the high early strength cements, combined with the humid air usually found in products plants, may cause caking of cement stored over too long a period and, as a result, considerably reduced strengths. This may be one of the reasons why the strength results with some high early strength cements are not as uniform as products manufacturers would like to see them.

SUMMARY

These tests indicate that for the method of curing used, (3 days moist room, then air until test) 70-lb. of high early strength cement gave compressive strengths higher than 94-lb. of normal portland cement at ages of 3 to 28 days. The difference in strength was more marked at the early ages (4 and 7 days) than at the 28-day period.

For such discussion of this paper as may develop readers are referred to "Supplement," JOURNAL *for Sept.-Oct. 1936. Discussion should reach the Secretary by Aug. 15, 1936.*

Discussion of papers presented by Committee 609:

"Placing Concrete by Means of Vibration"*

CONVENTION DISCUSSION

Ben Moreell (C. E. C., U. S. Navy): I should like information about frequencies. We have been told that a vibrator should place between 13 and 20 cu. yds. per hour. We see trade literature talking about frequencies of six, eight and ten thousand. Professor Davis has mentioned frequencies of eight and ten thousand volts on a bridge. What I want to know is whether, if we specify a vibration of say 8000 blows per minute, we can be sure that when that vibrator is buried in the concrete and has been working for some time, that vibrator will continue to operate at 8000 blows per minute? Has the equipment been developed to a point where we can be sure that if we specify a thing like that, the claims in the trade literature will be borne out?

A. E. Lindau (Chairman of Committee 609): The information is somewhat meager but we will have more shortly. Vibrators are on the market now that will measure the frequencies of the vibrator when immersed in concrete. The committee is not prepared now to answer that question specifically. However it is reasonable, I think, to expect that the frequencies talked about here of eight to ten thousand per minute, can be obtained because the power of the vibrators is such that they are not particularly slowed down in concrete. The slowing down in speed I have observed has not been over 15 per cent when the element has been immersed in concrete. However, that is pretty sketchy and is not a complete answer to your question. Have you something, Professor Davis, to add?

Raymond E. Davis (Univ. of California): I have nothing more definite to add. All I can say is that from observations of machines travelling at different frequencies and observations of machines travelling at different amplitudes, it seems as though we were headed in the direction of higher frequencies and smaller amplitudes.

*Ten individual papers, introduced by Chairman A. E. Lindau constituting the preliminary report of the Committee were published in three successive JOURNALS, March-April 1935, *Proceedings*, Vol. 31, p. 417; May-June 1935, *Proceedings*, Vol. 31, p. 527; Sept.-Oct. 1935, *Proceedings*, Vol. 32, p. 65.

A. S. Douglass (Detroit Edison Co., Detroit): I believe this subject is one of extraordinary importance. I have been impressed by the admitted character of the purely practical in these papers. I believe it is necessary that this whole subject be supplemented by laboratory investigation and determination. There are many variables which must be evaluated before such practices can be used and controlled effectively. Take for instance the matter of frequency and amplitude of vibration, I can imagine amplitude and frequency which will have the effect of fluffing up the mixture—somewhat the effect of a high speed cake beater making a light cake. Certainly such a result must be avoided in concrete. In fact, amplitudes and frequencies must be determined which will produce the opposite result as completely as possible. Some time ago in reading a paper on "Fashions in Engineering Above all Things" I was impressed by the fact that we are all, even we engineers, too inclined to rush into things. I believe too little is known about the effects of vibration at the present time, and I believe that if these papers convey to some contractor the belief that he can save all his concrete, whether sloppy, rich, lean or otherwise by vibrating it, the result may be most damaging. It should be borne in mind in respect to Professor Withey's test, that the whole body of concrete was vibrated, or thrown about, so to speak. Such a test might prove things which might not be at all true in a mass such as dam work. Therefore, while his test is a valuable contribution there is a clear necessity of going beyond it. In one of the papers it was stated that the vibrator should not be more than six inches from the form; auxiliary spading was also referred to. This definitely implies a limitation in the extent to which the mass is affected by the vibrator. Possibly core sampling and core testing as a laboratory follow-up may reveal the extent of the affected mass by various types of vibrations.

The remarks of Mr. Lindau have clearly indicated that these things are in mind and that the contributions at this meeting are progress contributions only. My purpose is to enter a word of caution against rushing too precipitately after a new fashion, and to proceed carefully in order that the very great promise which this matter holds may be fully realized.

Mr. Lindau: While I think Mr. Douglass has pointed out a feature of a paper that perhaps may not have become evident to the audience, certainly no one has been more aware of the possibility of an idea running away with us than your chairman, and in view of that fact we specifically asked these authors to give us the benefit of their experience completely, tell us what the troubles were as well as the advantages,

and you probably have noticed that in general they have done so. We no doubt can reach a point very readily where we think all that is necessary to get a perfectly fine piece of concrete is to get some vibrator and stick it in there and let it go. Well, most decidedly the view of the committee is that that is not the answer and the committee hopes to continue this work and to go along the line suggested by Mr. Douglass and get the fundamental basis for the use of vibrators so that we know very definitely what we are doing.

C. B. Gifford (*Concrete Plank Co., Jersey City, N. J.*): I have had some experience with concrete, more particularly in the vibration of three-course slabs. I have been making two-course concrete slabs about five years, and three course slabs about three years. We are using a very high speed. I find that if I have too much amplitude, that the mix becomes segregated, but if I have a high speed and little amplitude, I get a more dense and a stronger mix altogether. I have found in the use of a vibrator, that when the mixture I have in the pan (I am using a thin, three-course slab) has a creamy appearance on the surface, the mass is about the densest it will ever get. I get air pockets on the vertical surfaces; that is something I have not been able to overcome in an experimentation of about three years.

Commander Moreell: I just wanted to remark in reply to Mr. Douglass that with respect to frequencies: the vibrators whose use in France I reported, were started with a frequency of 900 blows per minute; that was stepped up to 1800 and a vast improvement in the product resulted. Later it was being stepped up, as I left France, to 3500. They felt that that increase in frequency had given them a vastly improved product.

John R. Nichols (*Boston*): A number of specifications have warned us against vibrating the forms. The vibration that was done in 1918 in building concrete ships was applied to the forms; they used just common forms, but it gave vibration and did consolidate the concrete. Why are we warned against vibrating the forms? In the Withey tests, the vibration was transmitted to the concrete through the form. Why should we be warned against that? It seems to me that it is an excellent way of transmitting vibration to the concrete.

Mr. Lindau: It is not so simple to answer that question, Mr. Nichols. I think more than one question is involved. The authors of these papers say you should not vibrate the form. As I get it, we have not had time to get their complete story. They have in mind the damage to the forms, but that is not the only thing involved; there has been a prejudice on the part of some people who have tried form vibration, so called; they believe that they get so much more tear to

the surface and that that is undesirable. However, in many cases that is the only thing you can do and it is the most practicable thing to do, and so far as damage to the forms is concerned, I might say that I know one whole job placed by vibrators on the basis of having the vibrator in contact with the forms almost the entire period of vibration, and those forms were used over and over again. It was a viaduct floor and the use of those forms was a considerable part of the economy in that project, and so far as I could observe, they were used about as many times as if they had had tamping. I could not see any particular damage to the forms. However, we cannot answer all the questions at once. We have had no opportunity to ask the authors' opinions about these matters, so we have presented the matter as it has come to us.

Professor Davis: I think there is another element there besides damage to the form. In the case of concrete ships, we were vibrating a very thin section. In other cases, where we vibrate the forms, we were attempting to consolidate a relatively thick section. I had in mind a job which I visited recently, a tunnel lining where both methods of vibration were employed. Now, when the forms were stripped, the surfaces, the interior surfaces of the tunnel looked about equally good, yet when cores were cut from that tunnel lining, it was obvious that the method of internal vibration had compacted the concrete for the entire thickness of the tunnel lining, whereas this form vibration had not done so. It seems to me therefore to be largely a question of the thickness of the section of concrete which it is desired to vibrate, and certainly in many cases form vibration, where we have thin sections and a larger amount of steel, is the only practical method.

H. S. Wright (Nazareth Cement Co., New York): One specialized form of external vibration, which I think the committee should investigate, is a matter of the vibration of so-called Lally columns. I am not familiar with their present practice, but about 1920 or 1921 they were in the habit of making a traprock concrete so dry and hard that it could not be compacted by any known means other than vibration. They placed it in the column which was from 4¾ in. to 10 or 12 in. in diameter, and with external vibration, pneumatic vibration, that concrete was compacted in the column until the water splashed from the surface. Cutting sections transversely of the longitudinal length of the column showed no indication of the aggregate, the coarse aggregate having been thrown away from the face of the column, and it showed a remarkable density along the entire length. I think it might be worth while to see what their practice is today and whether they have that same good result from it.

M. I. McCarty† (*Ludington, Mich., by letter*): Several papers have mentioned or recommended extreme frequencies ranging from 6000 to 10,000 r. p. m. Commander Moreell has asked enlightenment on this particular subject in order that engineers specifying vibration may outline frequency requirements intelligently.

This question deserves a reply by a manufacturer of vibratory machines. Upon manufacturers has rested the burden of introducing and perfecting the method and equipment for placing concrete by vibration. They must make available to the contractors machines that will perform satisfactorily and continuously under mechanical stresses that are serious problems in themselves. Every manufacturer of machinery, particularly of the high speed type, has spent untold sums of money in eliminating the very thing we are deliberately producing, to wit, vibration—the most destructive internal force in any piece of equipment.

This question of frequency is of such extreme importance and contains so much dynamite in its relation to the future use of the method that I feel it is necessary to call to your attention a few facts.

1. Practically all of the data available today, indicating the desirability of utilizing vibration as a distinct improvement in concrete placement, has been obtained with frequencies ranging from 3200 to 4000 r. p. m.

2. Vibration must be transmitted through the bearings of the machine; the lubrication is therefore a major problem. A lubricant that will operate a standard electric motor for nine months without changing will stand up approximately one week in a vibratory machine of the same frequency, this frequency being 3600 r. p. m.

3. In the three types of vibratory machines produced at this time, electric motors with unbalanced weights, vibratory units driven by flexible shafts and air turbine machines, the same limitations hold true. They can be operated for a short period at extreme frequencies but inevitably the life of the tool will be shortened, the contractor will be forced to carry more spares than should be necessary, and maintenance expense will be out of proportion to the advantage that may be gained through slightly accelerated placing speed. In fact, on the small jobs the contractor may be confronted with an expense which will preclude his use of vibratory equipment unless it can be added to the cost of the job which will defeat the very thing we are trying to accomplish, to wit, a better job for less money.

4. Extreme frequencies have been used on large projects mainly where the material is harsh and in most instances unusually dry in

†Electric Tamper & Equipment Co.

consistency. I distinctly question their use in thin sections where more workable concrete is required owing to the possibility of over vibration which will result in separation.

5. The question of sustained amplitude at varying frequencies is answered in the two pictures. Fig. 1 illustrates the amplitude of a 40-lb. vibrator at 3500 r. p. m. Fig. 2 illustrates the same machine operating at 5000 r. p. m. These amplitudes were obtained by attaching a stylus to the vibratory motor and drawing across the point of this stylus a celluloid strip, thus obtaining the direct amplitude of the machine and it will be noted that this remains practically the same regardless of frequency. A 90-lb. machine produces the same results with an increase in amplitude of approximately $\frac{1}{64}$ in.

In concluding this discussion I hope that the concrete industry will take advantage of and use the vibratory equipment available today with the assurance that the various manufacturers will improve and make additions to their lines as they can be reasonably expected to do in the course of competition and desire upon their parts to better their product.

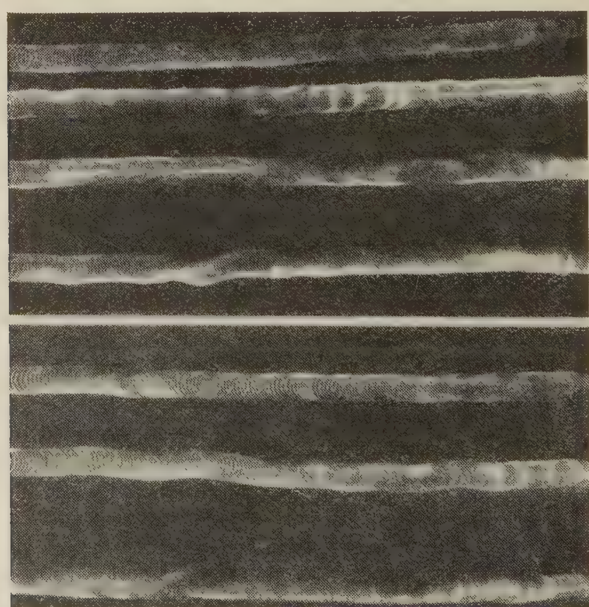

FIG. 1 AND 2—VIBRATION RECORD—3500 R. P. M. ABOVE—5000 R. P. M. BELOW

Albert Merciot (28 Rue Rousselet, Paris VII, France, by letter): Mr. Hathaway writes (A. C. I. JOURNAL, Mar.-Apr., 1935, *Proceedings*, Vol. 31, p. 422): that the best practice is to avoid putting the vibrator in contact with the reinforcing steel, particularly when the concrete must be put in place slowly and when the vibration might be transmitted through the agency of the steel to that portion of the concrete which has *already started to harden.*

Allow me to remark that it has been demonstrated that vibration of the elements of concrete in the process of hardening is not only without any danger but gives to this concrete an even higher degree of resistance than it would have otherwise.

Important research on this subject has been made by Mr. Giesek (Annual conference of the American Society of Standardization, 1921), and more recently by V. I. Soroker, engineer, Scientific Collaborator of the Institute of Scientific Research concerning Industrial Construction (Bol. Levchinsky per. 17 Moscow, USSR).

Since the foregoing discussion was completed Committee 609 has presented a further report (Recommendations for Placing Concrete by Vibration—A.C.I. JOURNAL, *March-April 1936, p. 445, and with that report as a basis, discussion is open until July 1.*

Discussion of papers by A. Burton Cohen, and R. B. Young:

"Supervision and Inspection of Concrete" and "Inspection"*

Charles C. McNamara (*Denver, Colo., by letter*): Messrs. Cohen, and Young have presented excellent papers on one of the most important phases of concrete work. They may well serve as the beginning of extended activity of Institute members in this important phase of concrete construction. During the last few years the extensive advancements made in the design of structures and in the knowledge of concrete properties have added immeasurably to the economy and applicability of concrete as a structural material. To realize their full benefits, inspection must advance with equal rapidity. Encouragement and assistance for such advance should be ranked as equal in importance with sponsoring of researches and building codes.

Purposely, the authors restricted their papers to only one phase of a broad subject. An adequate treatment of inspection and all its interrelated problems is more appropriately the duty of the whole Institute led by a select committee of experienced concrete engineers. Here is an unusually good opportunity for rendering distinguished service.

Many engineers and architects are fully aware of the inadequate inspection on many jobs, but they have neither the time nor the data required to "sell" the policy of good inspection to their superiors, or the owners. This fact, probably more than any other, results indirectly in poor or mediocre concrete, and makes necessary the relatively high factors of safety used in design. One of the best sources of "sales points" for inspection is the paper on the "Study of Defective Concrete" by F. R. McMillan (*Proceedings*, Amer. Concrete Inst., Vol. 27, p. 1039, (1931), but many valuable additional data are in other papers and in the experience of engineers all over the world. A compilation of such data and experiences prepared primarily to show the benefits of good inspection, and a discussion of convincing arguments in justification of inspection costs would undoubtedly be much used by engineers and architects throughout the world.

*Journal, Amer. Concrete Inst., Sept.-Oct. 1935; *Proceedings*, Vol. 32, p. 39.

A condensed, single publication of the principles of concrete construction and inspection would, by making reliable data and advice readily available, promote more and better concrete. Many manuals for concrete inspectors have been published by various organizations, but most of them are limited in scope and application to one type of construction or only one phase of the subject of inspection. Of necessity, such a publication should contain an extensive summary of the relationships known to exist in concrete, and recommended procedures for inspection. For the protection of the owner, the engineer, and the contractor, a list of subjects to be covered adequately in the specifications should be given, as a prerequisite to good construction. Recommended organizations, instructions, and sample record-forms in detail for relatively small jobs would be widely adopted in preference to the less satisfactory methods now used. Although a publication prepared to serve such a purpose would become out-of-date in time, many of the accepted principles of concrete construction will certainly remain unchanged, and only minor revisions would be necessary.

There should be a generally accepted handbook or manual of concrete inspection as a guide for inspectors and constructors just as there is an A. C. I. Building Code. The Institute by its many notable contributions to the advancement of concrete design and construction has become a recognized leader and authority in that field of work. An inspection manual sponsored by such an organization would be the foundation for standardization in construction and subsequent further advancements in the design and use of concrete structures.

Mr. Cohen's suggested qualifications for good inspectors are apparently far above those required by many executive engineers. In many organizations the inspection positions are by far the poorest. They frequently require unreasonably long hours of work at low wages —in some cases even lower than common laborers receive—and only seasonable employment is assured. Under such circumstances well qualified men will inevitably shun inspection and seek a more promising type of work.

The future prospects for better inspectors and better inspection will depend upon the attitude of those directing construction work. So that the problem resolves into one of convincing executive engineers, and owners that good inspection is well worth its cost.

EDITOR'S NOTE

Mr. McNamara's suggestions are good. The Institute's Advisory Committee anticipated the need which he mentions. An A. C. I. Manual of Concrete Inspection is in preparation.

Discussion of a paper by Davis, Kelly, Troxell and Davis:

"Properties of Mortars and Concretes Containing Portland-Puzzolan Cements"*

Bailey Tremper† (*Olympia, Wash., by letter*): The authors of this paper have made a partial presentation of the data that led them to the conclusions given and to their summary. On the basis of the reported data, the writer is unable to find justification for some of the statements made. It is felt that the authors properly should have given greater emphasis to certain results that might escape the attention of the busy reader.

For instance, the authors state in their summary that portland-puzzolan cements as a group produce concrete that is less permeable than is obtained with portland cements as a group. The data presented to substantiate this statement consist of tests of one portland-puzzolan cement only, compared to two portland cements. The data show total inflow into the concrete, not the amount passing through. Results are given for concretes containing 0.8 and 1.0 barrels of cement per cubic yard. It is noted that for the richer mix the reduction in inflow resulting from the use of portland-puzzolan cement is relatively much less. This may indicate that for cement contents greater than about 1.25 barrels per cubic yard, differences in permeability between the two types of cement might be negligible.

The authors' summary states: "It has been found that there is a fair correlation between the activity of a puzzolan (as determined by the compressive strength of puzzolan-lime-sand mortar) and the compressive strength at the later ages of mortar containing the corresponding portland-puzzolan mortar." Table 3 shows such a correlation for portland-puzzolan cements in *standard* mortar. The strengths obtained with plastic mortar are not proportional to those of standard mortar and the correlation between the former and the activity test is somewhat poorer. With respect to tests of concrete specimens, which in the final analysis are of greater importance than mortar tests, computations show that at ages of 3 months and 1 year most of the puzzolans contributed little to the strength. Of those showing high

*Journal, Amer. Concrete Inst., Sept.-Oct. 1935; *Proceedings*, Vol. 32, p. 80.
†Materials Engineer, State of Washington, Dept. of Highways.

activity only Clay C, calcined and oil impregnated diatomaceous shale, calcined were effective to the extent that the portland-puzzolan concretes attained strengths in excess of 90 per cent of the corresponding portland cement. Of the remainder (28-day compressive strengths in the activity test less than 800 p. s. i.), two puzzolans, calcined tuff and calcined pumicite, were effective to that extent.

The authors emphasize that the concrete in these tests was relatively rich. The writer does not believe that, when ¾-in. maximum size aggregate is used, concrete containing 1.50 bbls. of cement per cu. yd. is unduly rich, particularly if the concrete is to be subjected to aggressive waters.

It is noted that the activity test is conducted at a temperature of 130° F. It is well known that temperature has an important effect on the extent to which combinations between lime and silica take place. In the manufacture of sand-lime brick, reactions that would be negligible at normal temperatures are made to occur by means of temperatures in the neighborhood of 300° F. The portland-puzzolan mortar and concrete specimens made by the authors were cured for the first 28 days at 70° F. and thereafter at temperatures assumed to be not greatly in excess of this. It seems possible that the activity test might have had greater significance had it been conducted at 70° F. Moreover, the concrete in many structures, for which the use of portland-puzzolan cement might be considered as a result of the authors' recommendations, will not, in all probability, attain temperatures as high as 130° F.

The authors state in conclusion 26 that the "portland-puzzolan cements containing diatomaceous silicas, volcanic silicas, and limestones are more resistant to the action of sodium sulfate than are the corresponding portland cements, provided that the puzzolans are present in amounts greater than 10 per cent and up to 30 per cent, the limit of these tests." Exceptions to this generalization are found in two of the four materials in the volcanic silica group, namely basaltic tuff and Italian pozzuolana.

In their summary, the authors modify the above statement by the inclusion of activity and silica content as requirements for improved sulfate resistance. The basaltic tuff and Italian pozzuolana might be ruled out on the grounds that although active they were not sufficiently high in silica. The use of activity and silica content as criteria of sulfate resistance does not, however, appear to be entirely satisfactory with respect to the calcined clays. It is difficult to account for the pronounced difference in sulfate resistance exhibited by Clay A and Clay C on these bases.

The activity test has to do with reactions taking place between lime and silica. Gonnerman* in a study of cements of widely varying composition concluded that "of the four major compounds, tricalcium aluminate appeared to be the only one which reduced the resistance of mortar and concrete to 2 per cent solutions of magnesium and sodium sulfate." If this is true, it appears that for sulfate resistance, the essential requirement of puzzolans would be their ability to neutralize the effect of tricalcium aluminate or, more specifically, the hydrated form of this compound. It would seem pertinent to examine puzzolans for the possibilities of their aluminous compounds forming the destructive sulfo-aluminates. Tests for activity of a puzzolan with lime seem to be somewhat misdirected as a measure of sulfate resistance.

The authors state in the summary that "these investigations make it appear that portland-puzzolan cements of proper composition are suitable for . . . structures subjected to the action of aggressive waters." It is not found that adequate means of determining "suitable composition" have been presented in this paper. Attention is also called to the fact that the "aggressive waters" to which the portland-puzzolan mortars and concretes were exposed were relatively pure solutions of one salt only, sodium sulfate. With the exception of one inconclusive series, exposures were in the relatively constant temperature of the laboratory. The importance of tests by outdoor exposure to natural waters is well illustrated by the findings of Miller and Manson† who exposed concrete cylinders containing, as an admixture, 20 per cent of volcanic ash from northwestern Nebraska (silica content, 72.45 per cent) in a 1 per cent solution of sodium sulfate in the laboratory and in the sulfate waters of Medicine Lake, South Dakota. They found that "twenty per cent ash increased the resistance of laboratory cylinders more than 500 per cent . . . Twenty per cent ash did not, however, increase the resistance of the cylinders exposed in Medicine Lake; on the contrary it slightly reduced their resistance."

*Study of Cement Composition in Relation to Strength, Length Changes, Resistance to Sulfate Waters and to Freezing and Thawing of Mortars and Concrete, by H. F. Gonnerman, *Proceedings*, American Society for Testing Materials, Vol. 34, Part II (1934), p. 244.

†Laboratory and Field Tests of Concrete Exposed to the Action of Sulphate Waters by Dalton G. Miller and Philip W. Manson, U. S. Dept. of Agriculture Technical Bulletin No. 358.

Discussion of a paper by D. B. Steinman:

"Isteg Steel for Concrete Reinforcement"*

Bengt Friberg† (*St. Louis, Mo., by letter*): The paper presents interesting data in regard to structural advantages to be obtained by the use of reinforcing steel with high elastic limit. In all of the test beams reported by the author, the reinforcement consisted of either conventional type hot rolled structural grade bars or the Isteg type of reinforcement as produced by twisting, while restrained as to length, two structural grade plain bars. The increase in yield strength by reason of the coldworking of the individual bar in the latter case being from 41 to 52 per cent. It is undoubtedly to be expected that concrete beams reinforced with Isteg reinforcement would reflect this increased yield point in a corresponding increase in beam strength. Calling attention to any such strength increase is hardly required and is, of course, not peculiar to the Isteg type of reinforcement. Any further increase in strength with Isteg reinforcement, than that which is occasioned by the higher yield point, would, of course, be of great interest. Such information is not available from the tests as reported, which used conventional type reinforcing steel of a lower yield point for all comparative tests.

A picture of any possible strength advantage of Isteg reinforcement is obtained only from a comparison with conventional types of reinforcing bars of the same yield point. For that purpose, Fig. 1 has been prepared covering a great number of concrete beam tests reported in the technical literature, with tension failures recorded, and with reinforcing bars of varying known yield points. There have been included on this diagram values from Isteg reinforced concrete beam tests. Values on the inclined straight line in the diagram would indicate equality between reinforcing steel yield point strength and the calculated stress in the reinforcing material at the time of failure. The plotted results of the great number of tests included show the calculated steel stresses at failure generally well above the yield point strength of the reinforcing material. The diagram also shows the calculated Isteg reinforcement stress at failure well above the yield point strength

*Journal, Amer. Concrete Inst., Nov.-Dec., 1935, *Proceedings*, Vol. 32, p. 183.
†Laclede Steel Co.

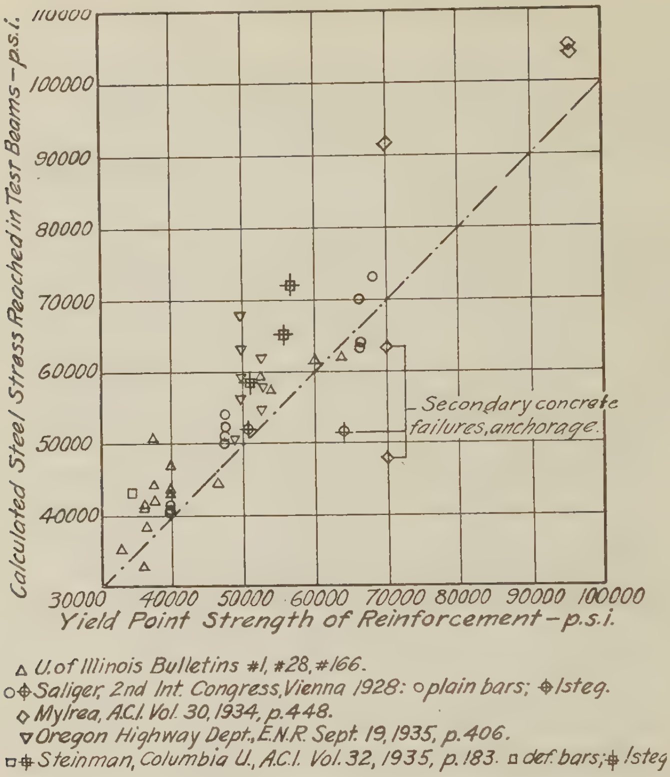

FIG. 1—RELATION BETWEEN STEEL YIELD POINT STRESS AND CALCULATED MAXIMUM STRESS REACHED AS REINFORCEMENT IN CONCRETE BEAM TESTS

of the material, but certainly no more than what has been experienced with conventional type reinforcement. The diagram shows clearly the great increase in strength to be gained in most concrete structures by using a higher elastic limit reinforcement.

It is quite normal that a reinforcing material, which by reason of coldworking has lost the clearly defined yield point characteristics of hot rolled low carbon steel, at the ultimate load on the conventional concrete beam will have a stress somewhat higher than the "laboratory-established" yield point stress. A similar delay in concrete beam failure may be exhibited by reinforcing steel with definite yielding characteristics, provided the mechanical bond between the steel and the concrete, because of surface roughness, is sufficient to confine the stretching at the yield point stress, actually existent at a tension crack,

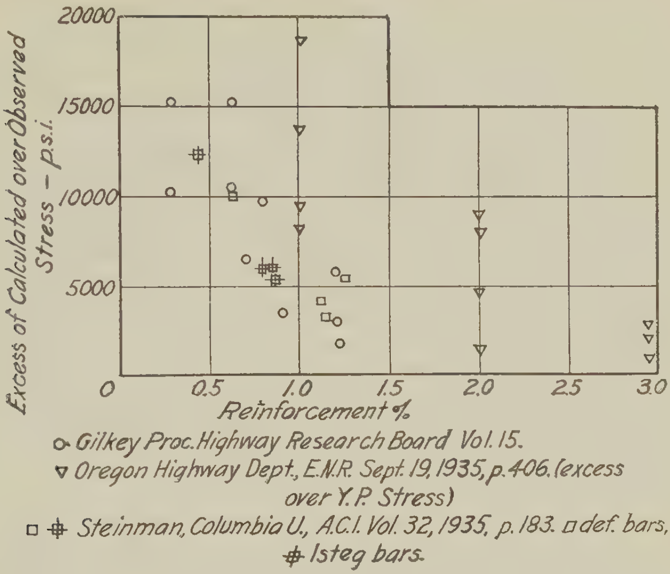

FIG. 2—EFFECT OF PERCENTAGE OF REINFORCEMENT UPON THE STRESS RELIEF IN TENSILE CONCRETE BEAM REINFORCEMENT

to a short length of the bar. In the Saliger tests, plotted in Fig. 1, plain round bars were used for reinforcement; and in these tests the maximum reinforcing steel stresses correspond most nearly to the yield point in the reinforcing. The concrete as a material is so brittle that it can hardly be expected to accommodate the stretching of the bar at the yield point over a considerable length without exhibiting ultimate distress. The Columbia University tests may not reflect the full delay that might seem possible if compared to the plotted data of tests made by Professor Mylrea with conventional type bars. Whether this is due to the peculiar characteristics of the reinforcing material, with nonuniform coldworking across the section, or to the routine of the tests is not certain.

The test data furnish an excellent comparison between the ultimate calculated steel stresses and the maximum observed steel stresses, which are lower than the calculated stresses and in each case approximately equal to the yield point stress, acting over the full gage length. There is evidence in recent research that the excess of the calculated over the observed steel stresses is dependent upon the percentage of reinforcement. The observed differences between calculated and observed steel stresses, plotted against the reinforcing steel percentage, are therefore shown in Fig. 2 for the tests as developed from the

author's tables, together with test data developed by Prof. H. J. Gilkey for other type of high elastic limit reinforcement and long-time load tests, published in the *Proceedings* of the Fifteenth Annual Meeting of the Highway Research Board (Dec. 4 to 6th, 1935).

For comparison and to indicate the recurrence of the trend, data furnished by beam tests in the Bridge Division of the Oregon Highway Department (*Engineering News Record*, Sept. 19, 1935, C. B. McCullough, author), including beams with distinct tension failure and those with calculated stresses higher than the yield point, have also been plotted. As the actual stress in the conventional type intermediate grade bars used was not observed, the stress relief in this series is assumed as the difference between calculated ultimate steel unit stress and the yield point stress for the reinforcement according to the Highway Department's test reports, ⅝-in. bars 49,600 p. s. i., four beams; ⅞-in. bars 53,025 p. s. i., four beams; 1-in. bars 47,925 p. s. i. three beams.

In all of these tests, the apparent relationship between the percentage of reinforcement and the possible stress relief in the reinforcing material is evident. The plotted data do not indicate any distinct difference in favor of one or the other type of reinforcement. The conclusion by the author that the higher calculated reinforcing overload with Isteg bars was due to inherent characteristics of that type of reinforcement is accordingly not conclusive. The greater calculated overload (stress relief) is rather a result of the smaller reinforcing percentage, and is possible with any high elastic limit reinforcement of satisfactory mechanical bond. It is suggested that the initial compressive stresses in the reinforcement, incidental to the shrinkage of the concrete in the test beams, provide the major portion of this stress relief. The experimental data so far available are not sufficient to indicate more than the distinct apparent relationship.

The author's statement that the Isteg type of reinforcement would have a beneficial effect in permitting overload on the concrete in compression seems based on the fact that a beam with Isteg reinforcement developed a computed concrete compressive stress of 3510 p. s. i., although the cylinder strength of the concrete was 3300 p. s. i. The statement is not distinctly supported by the evidence, as apparently none of the test beams were subject to concrete compression failure; and furthermore, experience of beam tests recorded in the technical literature generally indicates that such concrete compression failure would not be likely until the calculated stress exceeded the cylinder strength by a considerable margin, at least 25 to 50 per cent. For the

Oregon beam tests mentioned Mr. McCullough does not indicate concrete compression failures until the stress, calculated according to conventional methods, exceeded the cylinder strength by as much as 73 per cent (average of four tests). To this condition may be attributed in large measure the fact that generally the strength of the reinforcing steel determines the strength of most concrete beams under present design and construction practice. Any increase in elastic limit whether of Isteg or conventional types of reinforcement leads toward better balance of the design.

The bond test data have been presented in the paper in such manner that the true condition of bond stresses is not immediately discernible. These data have therefore been rearranged in Table 1 to show the

TABLE 1—BOND STRESSES AT FIRST SLIP OF THE TYPES OF REINFORCEMENT TESTED AT COLUMBIA U., A. C. I., VOL. 32, 1935, P. 183

Bar Size	Bond Area, 8 in. Length sq. in.	Stress in Steel p. s. i.	Load on Bar lb.	Bond at First Slip p. s. i.
ISTEG BARS				
⅜″ φφ	18.9	42000	9250	490
½″ φφ	25.1	28920	11560	460
⅝″ φφ	31.4	19040	11800	376
¾″ φφ	37.7	18960	16700	443
			Average	440
DEFORMED HOT ROLLED BARS				
⅝″ φ	15.7	23200	7200	458
⅞″ φ	22.0	14880	8900	405
1″ φ	25.2	14320	11300	448
1¼″ φ	31.4	10160	12500	399
			Average	435
PLAIN HOT ROLLED BARS				
⅝″ φ	15.7	17760	5500	350
¾″ φ	18.9	20000	8800	466
1″ φ	25.2	12160	9600	381
			Average	400

NOTE—8 in. embedment. Ultimate bond not reported.

actual comparative bond stresses, not the bond per linear inch of bar and the stress in the bar, only, as done by the author. The bond at first slip averages:

for the Isteg bars........................ 440 p. s. i.
for the deformed bars.................... 435 p. s. i.
for the plain bars......................... 400 p. s. i.

These results indicate that, insofar as first slip is concerned, Isteg reinforcement shows practically no advantage over other types of

reinforcement of equivalent size; that is, the pair of Isteg twisted bars compared with two single bars of the same size. Any impression of the greater bond resistance as a peculiar characteristic of Isteg twisted type of reinforcement is accordingly incorrect. The data only verify the fact that bond stresses decrease in proportion as the surface area of a reinforcing member increases. The twisting of the bars can only claim equality to the deformations in increasing the bond resistance at first slip according to these tests. The condition which the author's bond tables forcefully and correctly illustrate is the easy means of increasing bar stresses without increasing bond stresses by the self-evident expedient of decreasing the bar sizes. No concern about bond, so easily provided for, should need to hinder in correct design the utilization of high elastic limit reinforcing steel.

The Isteg reinforcement offers interesting possibilities worthy of consideration where large bond areas are required in a closely confined space. As a standard reinforcing material, and based upon the data presented, no advantage in strength can be claimed over conventional types of reinforcement of equivalent yield point. The excess unit cost of Isteg bars over conventional types of from 15 to 30 per cent would hardly be generally justified in construction, unless, as would seem to be the case in many localities in Europe, structural grade steel is the only conventional type reinforcing available at the cheaper price.

*Ben Moreell** (*Washington, D. C., by letter*): The development of Isteg steel reinforcing having come from Europe, it is interesting to know what experimental results have been obtained with that steel by European authorities.

In the "Report of the Building and Research Board for the year 1934" of the Department of Scientific and Industrial Research of Great Britain, an account is given of load tests on beams reinforced with ordinary mild steel and companion beams reinforced with Isteg steel. The following brief quotations from the report of the tests are pertinent:

In order to decide whether higher stresses are likely to prove satisfactory in practice it is necessary to consider their effect on the cracking and deflection that will occur, and the factor of safety against collapse. Cracking and deflection involve observations over a period of time sufficient to allow the effects of shrinkage and creep to develop.

Tests were made on four beams, two reinforced with mild steel and two with the special steel, in three stages:

(a) at working loads.
(b) at 50 per cent above working loads.
(c) increasing the load up to ultimate failure.

The working loads were maintained for a period of six weeks, the load then being increased to 50 per cent above the working load for a further period of six weeks.

*Commander C. E. C., U. S. Navy.

Throughout the whole period and subsequently up to failure, measurements of the deflections and crack widths were made on all the beams.

The results show that with regard to deflection and cracking due to strain the special steel behaves in much the same way as ordinary mild steel. At the same steel stress beams made with the two materials show similar deflections and cracking. At 27,000 p. s. i. the deflections of the beams with the special steel reinforcement were about 40 to 50 per cent greater than the deflections of the beams reinforced with ordinary steel at 20,000 p. s. i.

"There is, however, a marked increase in ultimate strength when the special steel is used in place of ordinary mild steel bar. At 27,000 p. s. i. the factor of safety for the special steel was 2.67, whilst for the ordinary mild steel the factor of safety at 18,000 p. s. i. was 2.33."

"At 27,000 p. s. i. the maximum recorded crack width was about 5×10^{-3} inches at the concrete surface as compared with 3×10^{-3} inches at a stress of 20,000 p. s. i."

The behavior of the Isteg steel, as far as deflection, width of cracks, and ultimate strength are concerned, is similar to that which would be expected with the use of high yield-point steel of the ordinary kind.

D. E. Parsons (Washington, D. C., by letter):* The author has performed a useful service in placing before American engineers a convenient source of information on the properties of Isteg steel and its value as reinforcement for concrete. If the primary purposes of the investigation and the paper were to present comparisons of the relative commercial values for European practice of different forms of reinforcement, the emphasis which the author places upon his estimates of efficiencies is partially justified by the data presented. Indeed the data appear to explain fully the wide interest abroad in the commercial utility of Isteg steel. However, values given for the relative efficiencies of Isteg steel and other forms of structural grade bars are not directly applicable to conditions in this country, because bars of structural grade steel rarely are used here as reinforcement.

Moreover, in interpreting the results of the tests of beams the author apparently overlooked existing information on the effect of amount of reinforcement on the strength of beams and on the compressive strength of concrete in flexure. The values given in the last column of Table 3 agree with data from other tests in that the factors of safety for beams which failed by yielding of the tensile reinforcement tend to become smaller as the percentage of reinforcement is increased.[1] Higher factors of safety and "superior margin of ultimate strength" for beams with Isteg steel, noted by the author (pp. 187 and 188), would be expected, therefore, because of differences in the amount of reinforcement in the beams, and should not be credited solely to unusual properties of Isteg steel. The finding of observed and computed

*National Bureau of Standards.
[1]Moments and Stresses in Slabs, by Westergaard and Slater, *Proceedings*, Amer. Concrete Inst., Vol. 17, p. 415, 1921. See especially Fig. 36.

stresses in the concrete which exceeded the compressive strength of concrete cylinders, as discussed on pp. 188, 189 and 191, also is not unusual but is common for beams which fail in compression and hence cannot be attributed to a peculiar behavior of beams reinforced with Isteg steel.[2]

When these facts are taken into consideration it becomes apparent that the use of higher working stresses in designing the beams with Isteg steel accounts, in part at least, for their higher factors of safety and the higher ratio of strengths as observed, to those estimated from calculations. Although the Isteg steel did not show a definite yield point, this property is not peculiar alone to Isteg steel but is exhibited by other forms of reinforcement, and it is not obvious from the data presented that similar beams reinforced with deformed bars having the same yield strength would have shown lower strengths than those containing the Isteg steel.

Despite the inapplicability to American practice of the author's method of estimating efficiencies from the results of tests of beams, it is believed that the information presented will prove to be of interest and value. The results of the bond tests indicate a stronger bond with Isteg steel than with the single bars and the observations on the distribution and size of tensile cracks in the beams seem to support those indications. These data indicate that Isteg steel may have advantage over other forms in this respect.

AUTHOR'S CLOSURE

As pointed out in the discussions, the behavior of concrete beams reinforced with Isteg bars can be but little superior to that of beams reinforced with other bars *of equal yield strength*, provided other essential characteristics associated with Isteg are also matched. With increased working stresses and decreased sectional areas of tensile steel, however, bond stresses rise. It is in this connection that Isteg bars offer additional superiority. By reason of their form and size, Isteg bars have a superior bond strength which more than compensates for the increased unit stresses at which they may be used. This is evident from the fact that two round bars with a combined sectional area equal to only two-thirds of a single round bar have 15 per cent more bonding area, and consequently at least 15 per cent more bond resistance. This relation is based on at least equal unit bond strengths for Isteg and ordinary round rods which, as the table in the discussion of Mr.

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[2]Compressive Strength of Concrete in Flexure, by Slater and Zipprodt, *Proceedings*, Amer. Concrete Inst., Vol. 16, p. 120, 1920.
Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams, by Slater and Lyse, *Proceedings*, Amer. Concrete Inst., Vol. 26, p. 831, 1930.
These papers, together with discussions, give an excellent description of the behavior of concrete beams which fail in bending.

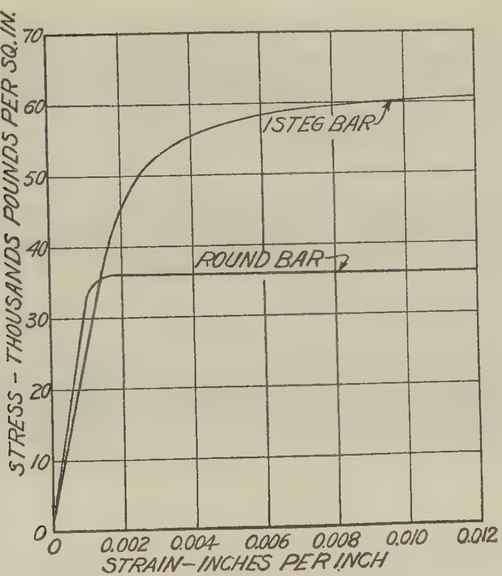


FIG. 1—STRESS-STRAIN CURVES

Friberg indicates, was generously substantiated by the Columbia University tests even if these test results are applied without corrective selection.

The bond tests made at Columbia University were such as to handicap the Isteg bars. The specimens were of the pull-out type and the bars were gripped about 15 in. from the face of the concrete. This arrangement subjected the bond to the combined effects of pull and twist. For this reason, only the tests on Group I are valid, and the tests on the larger sizes should be discarded. Subsequent comparative tests made in England in which the bars were either gripped close to the concrete or else totally enclosed in concrete beams gave for Isteg bars bond strengths of such high values that the omission of hooks has been officially authorized when Isteg bars are used.

In the recent English bond tests, ⅞ in. ϕ plain bars, *with hooks*, imbedded as reinforcement in concrete beams showed first slip (.005 in.) at a bond strength of only 210 p. s. i., and the bond failed completely when the tension stress in the steel was only 38,000 p. s. i., but ½ in. $\phi\phi$ Isteg steel *without hooks* in similar concrete beams under load showed no slip at all when the bond stress was 255 p. s. i. and the tension stress in the steel was 69,000 p. s. i. In pull-out tests on ½ in. $\phi\phi$ Isteg bars, the bars broke at 71,000 p. s. i. without any slip, the bond stress developed being 587 p. s. i. In terms of bond strength per square inch

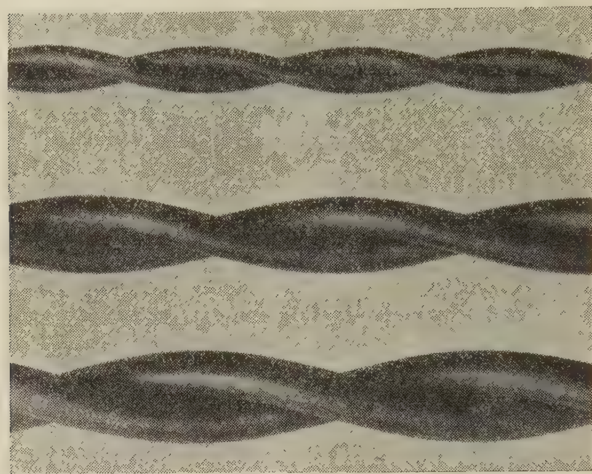


FIG. 2—ISTEG STEEL

of surface area, the Isteg bars showed a superiority of 180 per cent in pull-out tests; and a superiority of 21 per cent was retained in beam tests when the Isteg bars were without hooks and the plain bars were provided with hooks at the ends. In terms of tension stress in the steel at first slip (or without slip), the Isteg bars showed a bonding superiority of 86 per cent, which was reduced to 81 per cent when the Isteg bars alone were handicapped by the omission of hooks.

Comparative tests conducted in 1935 for the London County Council, on floor panels reinforced respectively with Isteg bars at 27,000 p. s. i. and plain structural grade bars at 18,000 p. s. i., showed no cracks visible in the plaster under either panel at full load. The panels were 13½ ft. square. The maximum panel deflection under full load after 24 hours was 0.048 in. for the panel with plain reinforcing and practically the same value, 0.049 in. for the panel with Isteg reinforcing. After removal of the load, neither floor showed permanent deflection to any practical extent.

Referring to Fig. 1 presented by Mr. Friberg, a comparison of slopes of straight lines from the origin drawn respectively through the plotted points representing the Columbia University tests on Isteg steel and through the plotted points representing Professor Mylrea's tests on high yield strength steel of conventional form, will reveal that, contrary to Mr. Friberg's statement, the Isteg steel produces a greater relative delay of concrete beam failure or a greater percentage margin of actual beam strength over theoretical strength.

Reference has been made in the discussions to the fact that beams with low percentage of tensile steel have relatively higher factors of safety. The reason is apparent when the behavior of a beam tested to destruction is noted. As load is progressively applied, the steel stress rises to its yield strength and then its rate of rise is arrested. The unit compressive stress in the concrete, however, continues to rise with increase of load. Consequently, the neutral axis rises, and the compression in the concrete is concentrated in a decreasing area until the ultimate strength of the concrete is reached. The rise of the neutral axis increases the effective value of the lever arm j, and this gives the extra margin of actual strength over the computed value. With a larger percentage of reinforcement, the ultimate strength of the concrete is reached at a lower position of the neutral axis. The increase in effective value of j above the value as conventionally calculated is therefore less and the margin of strength of the beam over the computed value is less. It is interesting to note that, in the Columbia University tests, the maximum extreme fibre stresses observed in the concrete beams in all cases checked the cylinder strengths very closely. It is the ultimate loads or calculated stresses at failure that materially exceeded the theoretical values in the beams with Isteg reinforcement, giving such beams a greater relative strength than indicated by conventional theory.

The fundamental thesis of this paper is that a beam reinforced with Isteg bars at a working stress of 27,000 p. s. i., together with a 15 per cent increase in concrete design stress, has a higher margin of strength and safety than a comparable beam reinforced with ordinary structural grade bars at 18,000 p. s. i., and the usual concrete design stress. This has been demonstrated. None of the discussions contradicts this conclusion.

The corresponding conclusion remains valid also for Isteg bars made of intermediate grade steel. When these are used, the design stress in the steel may be correspondingly increased from 20,000 to 30,000 p. s. i. In any case, when Isteg reinforcement of either structural grade or intermediate grade is used, the steel stress may be increased 50 per cent and the concrete stress may be increased 15 per cent, without reducing the value of n, and still yield reinforced concrete beams of greater strength and safety than beams with the conventional reinforcing.

Discussion of a paper by J. R. Shank:

"The Mechanics of Plastic Flow of Concrete"*

B. Fridenson† (*Moscow, USSR, by letter*): It gave me great pleasure to read the paper by Prof. Shank relative to the phenomenon of plastic flow. In the field of design, his work is no doubt a great step forward. I would however like to point out that in his conclusions there appears a rather doubtful point.

In a reinforced concrete column under the influence of continuous increment of deformation in the concrete due to its plastic flow, a continuous redistribution of stresses between the concrete and steel is constantly taking place.

The strain in concrete being composed of the initial loading strain and the strain due to plastic flow.

The experiments brought out the fact that the plastic flow is proportional to the stress, and in view of that in a reinforced concrete column plastic flow is taking place under changing conditions of stressing.

Mathematically expressed as follows:

$$e = \frac{f_{oc}}{E_{oc}} + \int_{o}^{t'} \frac{df'_c}{E'_c} . \ F\ (t) \qquad (1)$$

In this expression:

e = strain in concrete for the time t

f_{oc} = initial stress in concrete

E_{oc} = modulus of elasticity of concrete under instantaneous loading

E'_c = effective modulus of elasticity of concrete, equal to $\frac{df_c}{de}$

df_c = increment in stresses of concrete due to the infinitly small period of time dt that elapsed.

t = time under which column is loaded

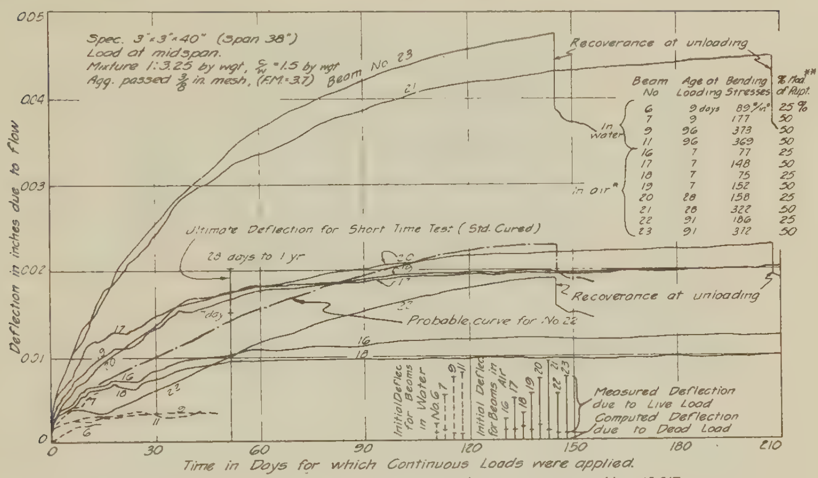
The stress $f_c = E'_c \ . \ e$ and

$$df_c = \left[\frac{\partial E'_c}{\partial t} \ . \ e + \frac{\partial e}{\partial t} . \ E'\right] dt$$

Substituting in the expression (1) we have:

$$e = \frac{f_{oc}}{E} + \int_{o}^{t'} . \ \frac{1}{E'}\left[\frac{\partial E'}{\partial t} . \ e + \frac{\partial e}{\partial t} . E'\right] F\ (t)\ \ dt \qquad (2)$$

*Journal, Amer. Concrete Inst., Nov.-Dec. 1935, *Proceedings*, Vol. 32, p. 149.
†Associate Professor of the Institute of Railway Engineering, Moscow.



* – Before loading, all beams were stored in water except Nos. 16 & 17 which were stored in air from the time they were taken out of the mold at 2 days.
** – Modulus of rupture determined from tests on companion specimens at time of loading.

We would easily be able to solve the differential equation (2) if we knew the law of change of E'_c and e depending on the time t due to the changing stress.

Unfortunately neither the formula of Professor Shank nor the formula of Thomas give this factor. From this follows that the formula advanced by Professor Shank is subject to doubt. Even if we introduce the changing modulus of elasticity M_{cx} it does not change the circumstances, because Shank's formula $Y = Ca\sqrt{x}$ can be accepted only under the conditions of permanent stress.

H. J. Gilkey† and George C. Ernst‡ (Ames, Ia., by letter): This paper and its companion publication [(30)]* supply such an admirable summary of the work done on plastic flow that the writers cannot resist the temptation to round out the record by appending certain other data not then available to the author. In fact, some of the more recent work is still in progress, and some of the data already obtained have not been analyzed fully. No striking differences in findings have been noted either in the data or in the interpretation of them, and definite support has been found for several of the author's statements as based upon his own observations or on citations from the work of others.

†Professor and Head of Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Iowa.
††Instructor in T. & A. M. and Junior Materials Engineer in the Engineering Experiment Station, Iowa State College.
*Numbers 1-30 inclusive refer to references cited by the author (p. 178) of his paper. Number 31 and upward supplement the author's bibliography and are appended at the end of this discussion. The numbering of tables and figures is also made consecutive with the numbering used by the author.

TABLE 5—SUPPLEMENTARY DATA FROM COMPRESSIVE PLASTIC FLOW TESTS (CONT. FROM TABLE 1 OF PAPER)

Spec. No.	By Whom Tested and Where	Unit Stresses		Age at Loading in Days	Conditions While L'd.		Cement Content By Weight		Kind of Aggregate		Fineness Modulus			Mod. of Elas. at Loading	Curve Values $y = C \sqrt[a]{x}$		Range of Observation In Days	Str'g't Part of Curve In Days
		p. s. i.	% Ult.		Rel. Hum. %	Temp. °F	Mix	c/w	Coarse	Fine	Coarse	Fine	Mixed		C	a		
112	Gilkey—Vogt U.S.B.R. Tests at Univ. of Colo. 1928-1929	175	25	3	40–60	70	1:3.25	1.5	None	Cr. Gr.	- -	3.7	3.7	2.19	0.63	3.4	0.01–240	1–100
113	"	350	50	3	"	"	"	"	"	"	—	"	"	2.06	0.46	3.1	"	4–100
114	"	375	25	7	"	"	"	"	"	"	—	"	"	2.09	0.25	2.2	"	3–100
115	"	750	50	7	"	"	"	"	"	"	—	"	"	"	0.21	1.9	"	1–100
116	"	375	25	7	Water	"	"	"	"	"	—	"	"	"	0.42	7.0	"	3–150
117	"	750	50	7	"	"	"	"	"	"	—	"	"	"	0.21	4.3	"	1–200
118	"	846	25	28	40–60	"	"	"	"	"	—	"	"	2.70	0.085	1.7	"	1–100
119	"	1544	50	28	"	"	"	"	"	"	—	"	"	2.50	0.052	1.55	"	1–100
120	"	800	25	28	Water	"	"	"	"	"	—	"	"	2.70	0.20	∞	"	1–200
121	"	1684	50	28	"	"	"	"	"	"	—	"	"	2.49	0.195	∞	"	1–200

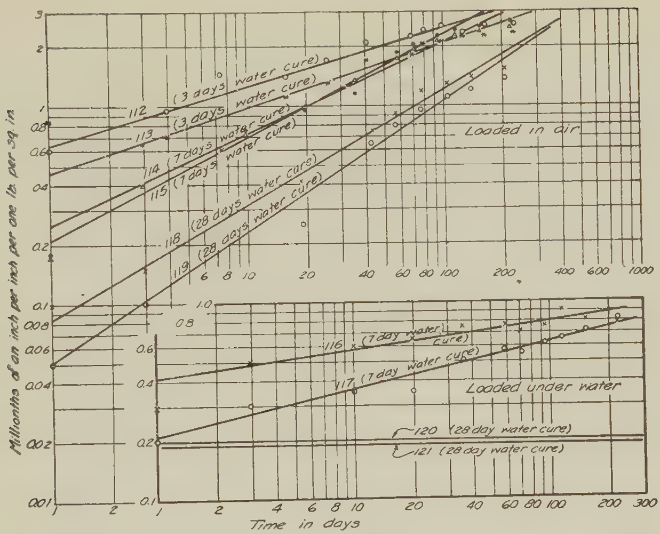


FIG. 11

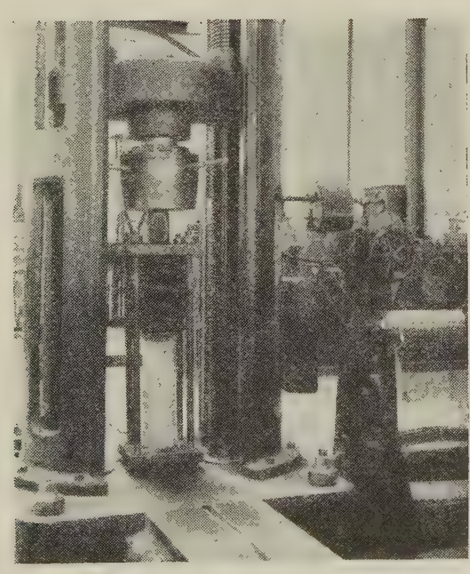


FIG. 12—APPLYING LOAD TO A 4 x 14-IN. SPECIMEN FOR COMPRESSIVE SUSTAINED LOADING TESTS

In August 1928, Dr. Frederik Vogt* who was then in this country in the employ of the U. S. Bureau of Reclamation, and H. J. Gilkey undertook a series of sustained loading tests on small plain concrete (virtually mortar) specimens subjected to compression, to flexure and to tension. The results of these tests have not been published fully, but summaries of some of the compressive and flexural flow data are available (31).

COMPRESSIVE SUSTAINED LOADING TESTS

The compressive flow tests were patterned after those of Davis (11, 12, 13, 14) which were then already under way. The specimens were 4 by 14-in. cylinders, being cast in molds borrowed from Professor Davis. Partial results are recorded in Table 5 as a continuation of the author's Table 1.

All the specimens, of whatever sort, were cast from identical batches and of the proportions and materials described in Table 5. The aggregate was crushed granite screenings from Stevenson Creek, California, the maximum size being ⅜ in. The mean ultimate strengths and moduli of elasticity of standard cured specimens tested in the usual manner are given in Table 6. No torsional sustained loading tests were made. Values for the properties of specimens subjected to other than standard curing that can serve as strength controls for specimens loaded in air are available (31) (32).

The values for the modulus of elasticity at loading in Table 5 are lower than the compressive value of Table 6. More time and jarring were involved in placing the sustained loads on the specimens than in a straight-forward compressive test which resulted in somewhat higher initial deformations. In other words, the values of Table 5 probably include some of the early time-yield.

TABLE 6—PROPERTIES OF THE GILKEY-VOGT CONCRETE
(All specimens standard cured by immersion)
See reference 31, p. 434, 461, 476, 502, 503, 504

Age	Strength (p. s. i.)				Mod. Elast. (millions p. s. i.)			
	Compr.	Flexure	Tension[1]	Torsion[1]	Compr.	Flexure	Tension[1]	Torsion[1]
7 days	1780	401	234	284	1.83	2.67	2.10	0.97
28 days	3200	570	275	413	2.69	2.91	2.65	0.99
3 mo.	4250	652	329	447	2.91	3.80	3.20	1.33
1 yr.	4850	631	333	504	3.32	3.03	3.05	1.58

[1]The data from tensile and torsional tests serve to supply additional information about the mixture used. The tensile flow tests were considered too few for generalization and were not published. (31 p. 509). Tensile flow tests are especially difficult because of the very limited stress (and strain) available prior to rupture. Torsional flow data could doubtless have been secured rather easily, but no effort was made to obtain them.

On Fig. 11 and in Table 5, the values of the constants appear to fall in line with those from others of the tests cited, except that the immersed specimens (No. 120 and 121) ceased to flow altogether after a

*Professor of Mechanics, Norges Tekniske Hoiskole, Trondhjem, Norway.

short time, giving horizontal lines on Fig. 11 and infinite values for "a" in Table 5. These indications are not entirely in agreement with the author's findings as summarized near the bottom of page 161 and in Table 3, although the difference is one of degree rather than kind.

Compressive flow specimens were loaded and unloaded in a testing machine after the manner of Davis, as shown in Fig. 12. A 3-rod cage, such as Davis used, would have given more leeway for taking the strain gage readings along the three longitudinal gage lines 120 degrees apart, and would have been preferable to the 4-rod cages that were used. The arrangement shown was satisfactory however.

FLEXURAL SUSTAINED LOADING TESTS ON SMALL PLAIN CONCRETE BEAMS

So far as known the tests of Fig. 13 represent the first and virtually the only recorded venture into the realm of plain concrete flexural tests of this kind [31]. As noted on the figure these beams were small 3 by 3 by 40-in. bars cast from the same batches as were the 4 by 14-in. compressive test specimens of Table 5 and Fig. 11 and 12.

More recently the writers cast and loaded two 3 by 6-in. by 10-ft. plain concrete beams which failed after two weeks at 50 per cent of the modulus of rupture of companion beams. The loading arrangement was such that the failure of one beam produced sufficient impact to bring failure to the other. These beams are reported as Beams No. 27 and 29. (33 Table 5.)

Referring again to Fig. 13, one may note that the beams loaded in water parallel the compressive specimens that were kept immersed under load. Unfortunately instrumental difficulties necessitated the discontinuance of the underwater tests after two weeks for Beams No. 6 and 7 and after a month and a half for beams 9 and 11.

Upon release of load there was a recovery roughly equal to the initial or elastic deflection with some slow additional recovery for a few succeeding days. Recoverance was not measured on the four immersed beams.

One should note that all of the Fig. 13 beams loaded in air were moist cured up to time of loading and that shrinkage was also occurring especially during the first few weeks under the load. Just how such shrinkage might have influenced deflections, no effort is made to say. The deflections as plotted do not include any attempt to correct for possible effect of shrinkage from drying out.

FLEXURAL SUSTAINED LOADING TESTS ON REINFORCED CONCRETE BEAMS

Rather complete data on twenty 3 by 6-in. by 10-ft. beams reinforced with varying amounts of steel and subjected to a wide range of steel

and concrete stresses are reported in [33] which was published several months after the author's paper. For the most part the findings are in good agreement with those of Professor Shank. Since the results are readily available [33] only a few of the more striking indications will be mentioned.

1. The flow of concrete under load in dry air is two or more times as great as that for concrete that is kept wet during the loaded period. This was true for both compression and flexure and apparently so for tension. Flow under water appears virtually to cease after a short time.

2. The Straub equation as originally proposed for compressive flow is only valid for a limited period as was pointed out by Vogt and Gilkey [34] and which has been further substantiated by the author.

3. For reinforced concrete beams loaded in air (even after months of air drying prior to loading) the shrinkage from continued drying is intimately related to the flow distortion because of the warping that results from the restraint of the reinforcing steel near one face, and no restraint near the other face. Warping was clearly shown for unloaded control beams containing off-center longitudinal bars.

4. For unreinforced concrete subjected to sustained flexural load, the writers are not prepared to state whether or not shrinkage effects the amount of flow deflection. Shrinkage was present in the plain concrete beams tested.

5. Shrinkage from drying out is intimately related to atmospheric humidity. The plastic flow of reinforced concrete beams slows down and even seems to cease altogether during periods of increased humidity (say 50 to 70 per cent), as during the summer months, only to be resumed when the humidity drops from turning on artificial heat or other cause.

6. There is a recovery approximately equal to the elastic portion of the strain upon the removal of load. This occurs at once. There appears to be a slight additional recovery that extends over a period of several days.

These are but a few of the findings that appear to be more or less common to the two investigations.

REFERENCES

References additional to No. 1 to 30 listed in the Bibliography at the end of the paper, JOURNAL, A. C. I., Vol. 7, No. 2 (Nov.-Dec. 1935), p. 178.

31. Engineering Foundation Arch Dam Investigation, Vol. II (May 1934), "Tests of Models of Arch Dams and Auxiliary Concrete Tests Conducted by the Bureau of Reclamation at the University of Colorado," by J. L. Savage, Ivan E.

Houk, H. J. Gilkey and Fredrik Vogt. (Published and distributed by The Engineering Foundation, 29 West 39th St., New York). pp. 518, 519. (Tables 64 and 65) and other portions.

32. Transactions Am. Soc. C. E., Vol. 100 (1935), p. 972, also *Proc.*, Am. Soc. C. E., Vol. 61, No. 1 (Jan. 1935), p. 131.

33. "Sustained Loading Tests on Slender Concrete Beams," by H. J. Gilkey and G. C. Ernst. *Proc.* Highway Research Board, Vol. 15 (1935), pp. 81-111.

34. Trans. Am. Soc. C. E., Vol. 95 (1931), pp. 699-703. Fredrik Vogt and H. J. Gilkey. See also *Proc.* Am. Soc. C. E., Aug. 1930 (Vol. 56, No. 6), pp. 1440-1444. The L. G. Straub paper, "Plastic Flow in Concrete Arches," of which this reference is a discussion is listed by the author as his reference No. 16.

Discussion of paper by R. E. Copeland:

"Load Performance Tests of Precast Joist-Precast Slab Floor Construction"*

W. K. Hatt (*Purdue University, by letter*): The Laboratory for Testing Materials of Purdue University has been attempting to develop a light precast joist for small house construction. Starting with the 8-inch joist designed by Mr. Copeland a comparison was made of two of these joists:

1. A joist with steel bonded to the concrete.

*Journal, Amer. Concrete Inst., Nov.-Dec. 1935, *Proceedings*, this Vol., p. 195.

TABLE 1

(a)

Joist	Test Measurements	External Load, (lb. per ft. of length) 0	40	80	160	200	240	
Prestressed	Deflection at Mid-span (Inches)	−0.129	−0.055	+0.005	+0.132	+0.197	+0.350	———
	f_s, p. s. i.	Ten. 2,760	Com. 1,000	Com. 4,900	13,200	17,400	22,400	Top of Beam
		Ten. 11,700	Ten. 13,500	Ten. 15,200	18,800	20,600	27,900	Bottom of Beam
	f_c, p. s. i.	Ten. 345	Ten. 60	Com. 240	Com. 900	Com. 1,250	Com. 1,975	Top of Beam
		Com. 905	Com. 610	Com. 325	Ten. 275	Ten. 560	Crack	Bottom of Beam
Conventional	Deflection at Mid-span (Inches)	——	+0.080	+0.155	+0.307	+0.383	+0.460	———
	f_s, p. s. i.	——	Com. 4,800	8,700	16,400	19,700	22,900	Top of Beam
		——	Ten. 3,400	7,800	17,400	22,000	26,600	Bottom of Beam
	f_c, p. s. i.	——	Com. 465	875	1,625	1,970	2,300	Top of Beam
		——	Ten. 340	Crack	——	——	——	Bottom of Beam

(b)

$$\text{Maximum allowable deflection} = \frac{\text{Span}}{360} = \frac{132}{360} = 0.366 \text{ inches}$$

Load required for 0.366″ deflection = {Prestressed Joist = 245 lb./ft. of length
{Conventional Joist = 190 lb./ft. of length

TABLE 1 (continued)

(c)

		Load—lb. per ft. Of Length	Mid-span Deflection (Inches)	f_s (lb./sq. in.) Top of Beam	f_s (lb./sq. in.) Bottom of Beam	f_c (lb. /sq. in.) Top of Beam	f_c (lb. /sq. in.) Bottom of Beam
Prestressed	First Tension Crack in Concrete	212	+ 0.211	18,190 Comp.	21,050 Ten.	1,352 Comp.	652 Ten.
	Failure	667	+ 2.32	—	— —	— -	— —
Conventional	First Tension Crack in Concrete	67	+ 0.126	7,480 Comp.	6,280 Ten.	729 Comp.	639 Ten.
	Failure	340	+ 0.649	30,800 Comp.	38,850 Ten.	3,119 Comp.	—

2. A joist in which the steel was prestressed in tension without bond with result that the concrete on the lower face of the joist was prestressed in compression.

Fig. 1 shows the design of the two joists. For the purpose of the comparison stirrups were omitted. Table 1 states the observations during the test.

The following comparisons result:

Length of tested span—11 ft.

Loading—third point.

Cement—tensile strength at 28 days—445 p. s. i.

Concrete—Haydite aggregate, 1:2½ mix.

Compressive strength—6,520 p. s. i., 28 days.

Ec—2,267,900 p. s. i.

Weight Joist—15½ lb. per foot run.

Weight Concrete—97 lb. per cu. ft.

Age of Joist at time of Test—44 days (28 days moist curing).

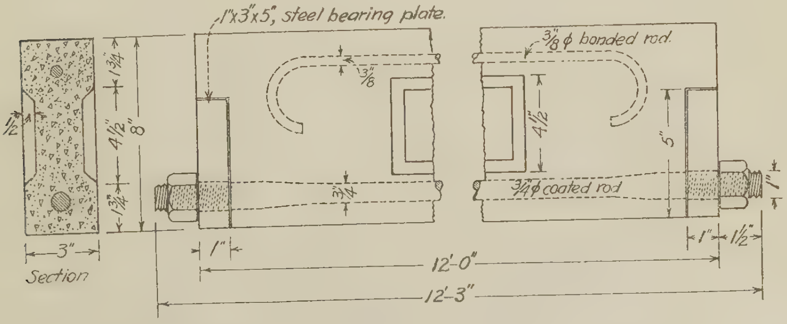


FIG. 1—DETAIL OF 3 x 8 x 12′ 0″ HAYDITE JOIST

Test Results	Bonded Steel Joist		Prestressed Joist	
	Zero Load	200 lbs. ft. run	Zero Load	200 lbs. ft. run
Steel f_s Top Bottom	—— ——	19,700 comp. 22,000 ten.	2,760 ten. 11,700 ten.	17,400 comp. 20,600 ten.
Concrete f_c Top Joist Bott. Joist	—— ——	1,970 top Cracked bottom	345 ten. top 905 comp. bottom	1,250 comp. 560 ten.
Load lbs. per ft. run at first tension crack	67		212	
at failure	340		667	
Tensile stress at first crack in concrete Bottom Top	639–*t* 729–*c*		652–*t* 1,352–*c*	
in steel Bottom Top	6,280–*t* 7,480–*c*		21,050–*t* 18,190–*c*	
Deflection Span ÷ 360 $= \frac{132}{360} = 0.366''$ at first crack lbs. per ft. run	190		245	

"Tentative Standard Specifications for the Design and Construction of Reinforced Concrete Chimneys"

(A. C. I. 505-36T)

"Proposed Specifications for the Design and Construction of Reinforced Concrete Chimneys" presented to the Institute by Committee 505, E. A. Dockstader, Author-Chairman, and published in *outline only* in Vol. 30, p. 367 (March-April 1934, Journal) were adopted unrevised as a tentative standard (505-36T) at the Institute's 32nd Annual Convention, Chicago, February 25-27, 1936. Copies of the complete report were sent on request to members.*

*Copies are available for general distribution at $1.25 each.

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Moscow subway

Engineering News-Record, Vol. 116, No. 15, April 9, 1936, p. 515-25. Reviewed by N. M. Newmark

In the first of two articles I. Gutmann describes the structure and construction methods of the Moscow Subway, and in the second article C. N. Pinco describes the stations and equipment. Seven miles of subway have been finished, requiring 3 million cu. yds. of excavation and over a million cu. yds. of concrete.

Past and present in asbestos cement production

E. Lechner, *Zement*, Vol. 25, No. 7, Feb. 13, 1936. Reviewed by Inge Lyse.

The cement products industry has only recently become familiar with the method of production used in the so-called asbestos cement products. This article describes the difficulties encountered in producing the best results and gives advice as to the proper procedure. Special attention is given to the cement pipe production because of its extensive use of asbestos. Different patented procedures are discussed.

Strengths of asbestos cement

Gerhart Rosenbaum, *Zement*, Vol. 25, No. 17, April 23, 1936, p. 292. Reviewed by Inge Lyse.

The expansion of the asbestos cement field from roofing material to more general use in cement products developed need for more information. The type of asbestos used was found to affect the strength of the cement to a considerable extent. The effect of the pressure used in the production was also found to be appreciable. Addition of powdered limestone, graphite, slate and talcum decreased the strength as did pigments for the colored products.

Light weight concrete construction in Berlin during 1935

F. K. Simon, *Zement*, Vol. 25, No. 8 and 9, Feb. 20 and 27, 1936. Reviewed by Inge Lyse.

Last year an intensive building construction was carried out in Berlin, partly because of government sponsorship. In the construction of individual homes light weight concrete was used extensively and also in group and apartment houses. This article presents a well illustrated review of the type of construction employed, as well as drawings for the different types of houses. Anyone interested in concrete house construction will find the article worthy of a thorough study.

Two-way reinforced concrete slabs partially loaded

L. Sussenberger, *Beton und Eisen*, Vol. 35, No. 3, Feb. 5, 1936, p. 45. Reviewed by Inge Lyse.

The author presents a theoretical analysis of freely supported, as well as partly

and fully fixed two-way slabs, loaded uniformly over portions of the slab area. Both symmetrical and non-symmetrical loading conditions are considered and final formulas are presented both for maximum moments and center deflections. Examples of design are also shown for the various conditions. Although admittedly approximate solutions, the author maintains that the results agree fairly well with more accurate methods of analysis for the more common cases of design.

Construction view points on vibrated concrete

R. B. WILLS (Engineer of Construction, State Highway Commission of Kansas). Highway Research Board (Dec. 1935). Reviewed by VERNE W. McCOWN.

Data for this paper were obtained from personal observation and study in the field and from reports submitted by various project engineers on construction work. The study includes internal vibrators on structures and vibrators on six pavement projects, covering work with mixed aggregates and coarse aggregates as used in highway construction in Kansas.

Generally, vibrators were entirely satisfactory, as the vibrated concrete had strength comparable with non-vibrated concrete having a greater cement content.

Through concrete trusses, 170 ft. long, used on low-cost highway bridge

W. E. BERRY AND GEO. RUNCIMAN, *Engineering News-Record*, Vol. 116, No. 1, Jan. 2, 1936, p. 1-4. Reviewed by N. M. NEWMARK.

The main span of the McMillin Bridge near Tacoma, Wash., is a reinforced concrete truss of 170 ft. span with a 22 ft. roadway. The contract price for the construction of the bridge was $36,000 which was cheaper than bids received for an alternate steel design with a 24 ft. roadway. The bridge was designed for H-15 loading with 30 per cent impact on floor slabs and 16 per cent on trusses. Details of the construction of the bridge are reported in the article.

Engineering applied to low-cost homes

Engineering News-Record, Vol. 116, No. 10, March 5, 1936, p. 349-54. Reviewed by N. M. NEWMARK.

Two examples are given of modern residence building in concrete. At Rochester, New York, a group of 43 single-family six-room houses with reinforced-concrete frame and floor slabs is being built at a total cost per house of $5,900, including land, grading, walks, streets, and utility installations. Features of the design and the construction methods making possible this low-cost are described.

The construction of a monolithic concrete house at New Orleans is described by D. S. MacBride in the second of the two articles. Particular attention was paid to form design and the methods used permitted a considerable saving.

Mohr's failure curve for concrete

ERNST BROD, *Beton und Eisen*, Vol. 35, No. 6, March 20, 1936, p. 104. Reviewed by INGE LYSE.

A review of the different experimental investigations of failure of concrete as compared with the Mohr curve. The French experiments by Considere as well as those by Caquot and Brice are discussed, as also are German experiments by Karman, Böcker and Ross, and American by Richart and Brandzaeg. As a conclusion, the author states that the French and American results were somewhat at variance, and since the French results agreed better with the Mohr's curve they are considered more important.

High strength materials in reinforced concrete beams

FRITZ VON EMPERGER, *Beton und Eisen*, Vol. 35, No. 4, Feb. 20, 1936, p. 60 Reviewed by INGE LYSE.

This discussion of high working stresses in reinforced concrete beams points out that experimental investigations have shown that with tension cracks in the concrete as wide as 0.3 mm. (about 1/100 in.) there has been no evidence of corrosion of the reinforcement. Attention is also called to the general requirement that when high yield-point steel with its corresponding high working stresses is used, a high strength concrete is also stipulated. Test results showing that the grade of concrete has little effect upon the strength of beams reinforced in accordance with general practice are presented as evidence of the fact that the yield-point strength of the reinforcement is the criterion of strength.

Effect of space framing in reinforced concrete construction

A. SCHERENZISS, *Beton und Eisen*, Vol. 35, No. 7, April 5, 1936, p. 121. Reviewed by INGE LYSE.

The realization of the continuity effect in reinforced concrete construction has recently brought forth numerous studies of the effect of torsional end rigidity on the bending moments of beams. In framed structures where beams are constructed monolithically with beams and girders, a considerable torsional moment may result from the bending of beams. The author attempts to analyze some particular problems to show that this framing effect may be significant in the design of reinforced concrete construction. Modern design requires more and more refinement in analysis and this article is a valuable contribution insofar as it calls attention to a field worthy of serious consideration.

Tests on fast and slow cooling of clinker

O. SCHWACHHEIN, *Zement*, Vol. 25, No. 17, April 23, 1936, p. 291. Reviewed by INGE LYSE.

A brief report of an investigation of the effect of fast and slow cooling of the clinker on the strength and chemical composition of the cement. The results showed that higher strengths at early ages were obtained both in tension and compression when fast cooling of the clinker was used. The difference, however, decreased with age at test so that at 28 days of combined curing the strengths were approximately equal. The fast cooling of clinker showed slightly higher trisilicate content than did the slow cooling and the specific gravities were 3.156 and 3.235 respectively. It is pointed out that the cooling of clinker is of small importance for the industries. The loss of heat with rapid cooling offsets the advantages gained in higher strength and easier grinding.

Designing concrete girder bridges for continuity

ALFRED BENESCH AND CHARLES E. MORGAN, *Engineering News-Record*, Vol. 116, No. 17, April 23, 1936, p. 604-6. Reviewed by N. M. NEWMARK.

The continuous concrete girder bridge crossing the Kankakee River near Wilmington, Ill., is described in this article. The bridge was designed and constructed by the Illinois division of highways and consists of three units of four continuous 83-ft. spans, totalling 1000 ft. in overall length, with a 44-ft. roadway and two 5-ft. sidewalks. The design was made for H-15 live loading with working stresses of 1200 p. s. i. in concrete and 18,000 p. s. i. in the reinforcing steel. Cylinder strengths at 28 days were about 5,000 p. s. i. The cost of the bridge was $189,000. A brief discussion of the methods and formulas used in the design of the continuous girders is included in the article.

Concrete proportioning

R. DANTINNE AND R. JACQUEMIN, University of Liege, *Bulletin No. 22*. Reviewed by R. L. BERTIN.

The paper treats of concrete proportioning from the standpoint of the most rational and economical use of the binder. The authors discuss the various factors having an

influence on the quality of the finished product. They bring out in particular the disadvantages of using an excess of fines in the aggregate mixture. For ordinary concrete, economy lies in the judicious use of available materials of good quality. A practical and rapid method of proportioning, based on the use of the Fuller-Bolomey curves, combined with Abrams' fineness modulus is given and applied to actual examples. Another example dealing with a very high quality concrete and high density shows the weaknesses of methods of proportioning based on granulometric curves, or by the fineness modulus, for such concrete.

SO_3 content in portland cement clinker

G. Mussgnung, *Zement*, Vol. 25, No. 15 and 16, April 9 and 16, 1936. Reviewed by Inge Lyse.

This paper presents a discussion of the SO_3 contents in the different raw materials for cement and their effect upon the chemical composition of the clinker. These are among the findings of the author: Contents of more than two per cent have no detrimental effect upon the qualities of the cement, but the lime content of the clinker is generally lowered. The SO_3 content decreases in direct proportion to the increase in temperature in the kiln. For the type of clinker investigated, the time of set could be regulated by the SO_3 content; for a content of 1.8 per cent or more the cement was slow setting. The SO_3 content as with gypsum has a beneficial effect upon the volumetric behavior while for strength properties gypsum and SO_3 do not act the same. The heat of hydration is only slightly affected by the SO_3 content.

Pickwick landing concrete plant

Herbert F. Gough, *Engineering News-Record*, Vol. 116, No. 16, April 16, 1936, p. 552-4. Reviewed by N. M. Newmark.

The Tennessee Valley Authority's Pickwick Landing Dam is an earth dam the major part of which will be constructed by the hydraulic-fill method. The concrete spillway, powerhouse, and lock structures required an extensive plant for aggregate handling and concrete mixing. A description of this plant, which is now placing concrete at the rate of 144 cu. yd. per hour, is contained in this article. Concrete aggregates are moved from barges to stockpile and from there to the mixing plant by belt conveyors. Cement is stored in a 6,000-bbl. elevated steel silo by pump. Three 2-yd. mixers are used in the mixing plant. Batching and mixing are controlled by electric-light signal systems. A detailed explanation of the various ways of delivering the concrete to the forms is also included in the article.

Compressive strength, impact and wear of concrete

A. Guttmann and K. Seidel, *Zement*, Vol. 25, No. 14, April 2, 1936, p. 233. Reviewed by Inge Lyse.

This report from the research laboratory of the association of Eisen portland cement manufacturers presents the results of an investigation of the qualities of concrete. Among interesting results should be mentioned the indication of increase in impact value with increase in compressive strength. Crushed stone concrete gave higher impact values than did gravel concrete of equal cement content and gradation of aggregate, although the compressive strengths were essentially the same. The wear resistance of gravel concrete increased also with the increase in compressive strength. For the type of crushed stone used, the wear resistance was less than for gravel concrete. The effect of the type of the cement was similar for all three tests, that is, a cement giving high strength will also give high resistance to impact and wear.

Plastic mortar tests for highway cement

OTTO GRAF, *Zement*, Vol. 25, No. 7, Feb. 13, 1936. Reviewed by INGE LYSE.

To develop a dependable test method, an investigation was made of a number of factors which might affect the results, including construction and preparation of the forms for the specimens, amount of water for equal consistency and variation in consistency for equal water content, method of placing the mortar in the form, type of dry curing, effect of size of compressed area on strength results, variation between individual results at the same laboratory, and between different laboratories and the correlation between the mortar strength and concrete strength. Each factor has been analyzed carefully and the conclusions with substantiating data are given. Without going into the details of Professor Graf's conclusions, it should be pointed out that the plastic mortar tests are found superior to the standard tension tests for determination of the quality of the concrete.

The old n-method and the new n-free method of designing reinforced concrete beams

FRANZ GEBAUER, *Beton und Eisen*, Vol. 35, No. 2, Jan. 20, 1936, p. 29. Reviewed by INGE LYSE.

This thorough discussion of the many different proposals for the elimination of the so-called n-method of designing reinforced concrete beams is a timely contribution to modern ideas. At first considerable space is devoted to the basis and the practical limitations of the orthodox reinforced concrete design in which the value n is defined as the moduli ratio. Next a review is presented of the various substitutes proposed in recent years for the elimination of n. The proposals by Stüssi (1932), Steuermann (1933), Saliger (1933), Gebauer (1934), Saliger (1935) and Bittner (1935) are discussed at length. All these new proposals have the one common basis that the compressive strength of the concrete and the yield-point strength of the reinforcement are the criterions for design, otherwise the proposals are at considerable variance.

Vibrating pan-type finishing machine for concrete pavements

F. V. REAGEL AND T. F. WILLIS (Missouri State Highway Department). Highway Research Board (Dec. 1935). Reviewed by VERNE W. McCOWN.

For the purpose of investigating the efficacy of the vibrating pan type of finishing machine, one of these machines was used in the construction of 4 miles of 20 foot concrete pavement. Two series of concrete mixtures were used. In one the cement factor was kept constant while the ratio of fine to coarse aggregate was varied; in the other series the cement factor was varied. Beams, 2 ft. by 5 ft. were removed from the pavement, tested for flexural strength and observed for honeycombing, and tested for density. One set was then tested for compressive strength and the other set subjected to freezing and thawing in an attempt to determine relative durability.

The results indicate that this type of machine can finish concrete mixtures which are much leaner, harsher and drier than mixtures ordinarily considered suitable for pavement concrete.

Thermal properties of concrete construction

F. B. ROWLEY, A. B. ALGREN AND CLIFFORD CARLSON, *Heating, Piping and Air Conditioning*, Jan. 1936. Reviewed by R. E. COPELAND.

This article presents the results of recent tests of various types of concrete masonry and monolithic concrete walls. The effect of different factors such as wall thickness, air space design, kind of aggregate and supplementary insulating materials is reported. In general, the thermal conductivity of the concrete varied with its density. Plain walls were improved by adding air spaces such as by furring, by using an insulated type of plaster base, by filling the air spaces with lightweight materials such

as granulated cork and rock wool. The heat loss through a cinder block wall was reduced about 50 per cent by filling the core spaces with rock wool. This is about the same reduction as is obtained with ½-in. plaster on ½-in. rigid insulation board furred out on wood strips. Overall heat transmission coefficients are given so that the relative efficiency of the different walls and supplementary insulating materials may be compared.

Concrete and the construction of large dams

M. RENE MARTIN (Engineer of Ponts et Chaussées), *Les Annales des Ponts et Chaussées*, Vol. IV, April 1935. Reviewed by B. MOREELL.

The author discusses construction procedures including selection of the aggregates, mixtures, transporting and placing the concrete. He does not touch upon the question of characteristics of the cement which is receiving so much attention in America. He emphasizes the great importance of "workability" (a word which the French engineers have borrowed from America). However, he feels that the utility of vibration, while existent, has been over-emphasized in mass work. His argument is not convincing and not in accord with American experience. He favors the use of an admixture to promote workability; in this case, it is Kieselguhr. He states that the granulometric composition of the aggregates is of equal importance with the water-cement ratio in obtaining good concrete on the job. He states that the usual factors of safety in mass concrete design are too large and experience justifies a less conservative practice. This article is an interesting summary of French opinion on concrete dam construction and is well worth the attention of American engineers.

New light weight aggregate

Concrete Bldg. and Concr. Products, Vol. XI, No. 4, April 1936, p. 69-70. Reviewed by J. C. PEARSON.

"Foamed Slag" is the name of a new light weight aggregate recently introduced into England. It is made by a patented process of rapidly cooling molten slag, and is said to produce an amorphous glassy and highly cellular material, low in sulphur content and absorption, but with marked hydraulic properties. The weight per cubic foot for various sizes is given as follows: ¾ to ½ in., 22 to 24 lb.; ½ to ⅛ in. 28 to 30 lb.; ⅛ in. to dust, 40 to 43 lb. The compressive strengths of foamed slag concrete (28 days) varies more or less directly with the density from about 350 p. s. i. at 65 p. c. f. to 2200 p. s. i. at 105 p. c. f. A marked increase in strength is reported between 28 and 90 days, which is attributed to puzzolanic action. The principal field seems to be in the production of non-load bearing building blocks of good heat-insulating quality. Comparative values of thermal conductivity (B. T. U. per sq. ft. per hour per deg. F. temperature difference between opposite faces) for two other wall types are as follows: Fletton bricks in cement mortar, 0.79; hollow pumice-concrete blocks, 0.42; 1:12 mix foamed slag panel rendered (stuccoed) on one face and plastered on the other, 0.29.

Use of vibration in placing pavement concrete—Ohio experience

R. R. LITEHISER (Chief Engineer, Bureau of Tests, Ohio Department of Highways). Highway Research Board (Dec. 1935). Reviewed by VERNE W. McCOWN.

A vibratory finisher was used on two experimental projects in 1930 and 1933. Four miles were then built under contract in 1935, two miles being finished with a Lakewood machine and two miles with a Blaw-Knox. The two finishers gave equally satisfactory results. Specifications were changed in some respects from those for the regular concrete pavements: forms were required to have a minimum base of 8 in.

instead of 6 in.; fine aggregate (by volume) was 28 to 36 per cent instead of 30 to 40; slump ½ in. to 1 in. instead of 1 in. to 2½ in.

Data on the relations between vibration, compressive and transverse strength, consistency and cement factor are presented. Experience on this contract demonstrated that by this vibratory method concrete could be placed with lower water-cement ratio and less sand content than by methods in common use, with accompanying increase in 7-day compressive strength. Test specimens made by the standard rodding method were found to be representative of the vibrated concrete. Three different brands of cement were used. The strength of concrete containing one of these was consistently lower than that containing the other two brands.

Light weight aggregate produced from slate waste

E. H. COLEMAN, *Concrete Bldg. and Concrete Products,* Vol. 11, No. 3, March 1936, p. 47-50.
Reviewed by J. C. PEARSON.

When certain kinds of slates are heated to a sufficient temperature, they lose the characteristics of slate and expand to many times the original thickness with formation of cavities. To perform in this manner, the composition of the slate must permit incipient fusion just before the evolution of gases. Slates suitable for light weight aggregate should expand from 3 to 7 times the original thickness, from the heat treatment. The expansion occurs perpendicular to the direction of cleavage, and a nodule of the treated slate when broken across shows a mass of small cells separated by glassy walls, and an impervious vitrified skin on the outside. In the production of light weight aggregate, the slate is crushed to the proper size, and burned in a rotary kiln to produce the vitrified nodules. Thus a more impervious aggregate is obtained than by crushing large lumps after burning. Nevertheless the crushed aggregate is only very slowly pervious to water, days or even weeks being required for light pieces to absorb enough water to cause them to sink. The important properties of light weight concretes are listed as: lightness, volume stability with change of moisture content, freedom from efflorescent salts, protective action on imbedded metals, sawing or nailing ability, resistance to passage of heat and sound. The excellence of expanded slate concrete is shown by a tabulation of its properties in comparison with those of concretes containing other light weight aggregates.

Method of computing the resistance of continuous bents and frames

HENRI BORDIER, *Le Genie Civil,* Vol. CVIII, No. 7, Feb. 15, 1936, p. 153-56.
Reviewed by R. L. BERTIN.

Following an article published in *LeGenie Civil* of March 15, 1924, page 248, giving a general method of solving rectangular frames consisting of beams and columns based on the determination of the fixed points of the constituting members, the author in his article restates the basic equations for determining the fixed points and develops certain simplifications which by means of numerous examples he demonstrates result in values very close to the true ones. Many equations are given for the moments at the supports and in the midsections for various types of structures and loadings which are based on relative stiffness factors and expressions for the type of loading under consideration. A graph is given which gives the negative moment coefficients for uniformly loaded members for varying values of the right and left fixed points. In conclusion, the author states that the methods described in the article are characterized as follows:

(1) Rapid means of obtaining the fixed points.

(2) Determination of the support moments for the loaded span by means of well

known formulas with simplification through the use of a graph in the case of uniform loads.

(3) Determination of moment propagation.

(4) In most cases even when complex, with judicious assumptions simplification of the fixed points adjacent to the supports or determination of the maximum positive and negative moments without recourse to the fixed points or giving consideration to the moment propagation from adjoining spans.

Studies of the accelerated soundness tests

Leo V. Garrity (Laboratory Supervisor, Michigan State Highway Laboratory) and Herbert F. Kriege, (In Charge, France Stone Co. Laboratories). Highway Research Board (Dec. 1935).
Reviewed by Verne W. McCown.

The sodium and magnesium sulfate soundness tests have been accused of unreliability because of difficulties experienced in obtaining check results between several laboratories or within one laboratory using the same materials and test methods. This condition justifies a critical study of the test technique. Beginning with the theory of the test, the authors have given attention to the several steps in the preparation of the sulfate solutions and the specimens to be tested, and the care necessary to keep the solutions at their proper concentrations. New data are given for the dissolution rates of the hydrous and the anhydrous forms of both sulfates, together with a review of the literature dealing with the solubilities of these salts. Crystallographic data are also given to aid in the easy identification of the crystal forms which may appear under the conditions of the soundness test. Illustrations are given showing the observed crystals and thin rates of growth.

Such factors as time of drying, temperature variations, aggregate containers, etc., on the constancy of results are considered, with supporting evidence. Distilled water as an immersing fluid is compared with sodium and magnesium sulfates. The constancy of check determinations with materials from several sources is shown. The interpretation of the present test is discussed and certain changes are recommended.

The testing of fine and coarse mineral aggregates by the accelerated soundness methods is discussed frankly in view of field observations and other established facts. Several recommendations are made with the hope of fixing the status of the test. A bibliography is appended which includes the most important chemical references as well as the researches in the realm of the test engineers.

Mustapha Breakwater at the Port of Algeria

M. Pierre Renaud (Engineer-in-Chief of Ponts et Chaussées, Director of the Port of Algeria), *Les Annales des Ponts et Chaussees*, Vol. IV, April 1935.

Jetty of Mustapha at the Port of Algiers

M. Pierre, J. M. Renaud (Engineer in Chief of Ponts et Chaussées, Director of the Port of Algiers), *Les Annales Des Ponts Et Chaussees*, May 1935.
Reviewed by B. Moreell.

The complete failure of a large section of the Mustapha concrete breakwater in the violent storm of February 3, 1934, led to this study of the causes of the failure and desirable changes in the design of this type of structure. This breakwater was considered the last word in conservative design of the "vertical wall" type. The preliminary studies were carried out over a period of years and every known precaution was taken to prevent failure. In this article, the author describes the characteristics of the structure and recounts the story of the failure. In a subsequent article, he proposes to discuss: (a) desirable changes in the design, and (b) the practical limits governing the use of vertical wall breakwaters and the circumstances under which breakwaters with a sloping face are preferable.

In the second article the author discusses in detail the significance of the positions in which the parts of the destroyed breakwater were found after the storm and draws

conclusions as to the cause and manner of failure. Soundings were made by mechanical and electrical methods to determine the positions of the submerged parts of the destroyed structure. Measurements were previously made to determine wave pressures against the breakwater. These were later confirmed by measurements made during the storm. Model tests were made to verify the theoretical calculations and measurements made on the actual breakwater. The work included exhaustive studies of the structural and hydrodynamic features of the design. The author lists his conclusions in a logical and comprehensive manner. The article is well worth the careful study of engineers interested in water-front structures.

Initial stress in reinforced concrete

H. KAYSER, *Beton und Eisen*, Vol. 35, No. 1, Jan. 5, 1936, p. 14. Reviewed by INGE LYSE.

In this report from the experimental laboratory of the Technical University of Darmstadt a detailed description is given of an investigation of shrinkage stresses in reinforced concrete structures. The investigation was a study of the effect of length of the reinforced concrete member and the percentage of reinforcement on the shrinkage stresses. Theoretical consideration has been given to the evaluation of the test results and the test data are summarized both with respect to total shrinkage and resulting stresses in steel and concrete. The outstanding results were:

1. The initial (shrinkage) stress in the reinforcement was greater for long specimens than for short, and greater for low percentage of reinforcement than for high percentage. For long specimens with low percentage of reinforcement the yield-point stress of the steel may be reached.

2. The shrinkage stress in the concrete increased with the length of the specimen and with the percentage of reinforcement. The tension strength of the concrete was exceeded even in relatively short specimens having an ordinary percentage of reinforcement. The danger of shrinkage cracks is therefore present.

3. The bond stresses which resulted from the shrinkage were practically independent of length of specimen and percentage of reinforcement. These stresses were below the bond strength values. The author makes the following observations regarding the effect of these shrinkage stresses upon reinforced concrete structures:

(a) Shrinkage cracks may be expected in highly reinforced concrete columns.

(b) The shrinkage will promote the formation of tension cracks in the reinforced concrete beams and may therefore increase the danger of corrosion of the reinforcement.

(c) For concrete highway slabs it is recommended to use no reinforcement on solid sub-grade and a minimum of reinforcement on yielding sub-grade in order to minimize the cracks in the pavement.

(d) Welded reinforcement is recommended because of the greater tendency of shrinkage cracks where lapped reinforcement is used.

Contribution to the study of the vibration of concrete

HERMITE AND MARIANI, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, Vol. 1, No. 1, p. 18-24, 1936. Reviewed by P. H. BATES.

This article is recommended to all interested in the vibration of concrete. Unfortunately, it lends itself poorly to brief abstracting. There is so much of interest that it is worthy of being presented almost in full.

A theory of the nature of concrete is first presented which is so developed that it shows the need of close contact of particles to bring about optimum results. Since the ideal concrete, according to this, would be so dry as not to be placeable, the authors bring out compromises which must be made to reach the desired optimum and the

mechanics of placing such "compromise concrete." This leads to an exposition of what vibration must do—impart a *certain amount* of *energy* to the concrete in order to move certain parts of it from one position in the mass to another. Since the concrete is heterogeneous, that is, composed of large and fine particles, theoretically different amounts of energy should be applied to the different sized particles. Since the required energy is not defined by frequency of vibration alone (as discussers of vibration in the United States would seem to indicate—reviewer's comment), the authors bring into the discussion the other essentials, namely, the mass of the vibrating agent and its amplitude. The theory calls for a possible reduction of energy during vibration to insure maximum compaction in minimum time with minimum segregation, particularly if there is a marked difference in particle size.

Two devices are shown for automatically determining and registering the degree of compaction and the frequency and amplitude of laboratory vibrated specimens. Examples of curves obtained are shown and some discussion of the effect of the amount of water and paste on the compaction is given.

An outline of further research now under way is also presented. Some conclusions based upon incomplete work show that without vibration maximum compaction can be attained but through the use of a quantity of water which would yield poor strength. Maximum compaction is reached by vibration with considerably less water than when vibration is not used. The amount of water giving best compressive strength is not the same amount that gives maximum compaction by vibration.

Single bent rigid frames with offset supports

ALEXANDRE SPOLIANSKY, University of Liege, *Bulletin No. 28.* Reviewed by R. L. BERTIN.

This paper is a study of the possibilities which are offered by offsetting the supports of single bent rigid frames first inwardly and second outwardly from the axis of the vertical members. Formulas for moments and thrusts are derived for single bents with hinged offset supports both inwardly and outwardly and for one and two stories. For some of the cases the moments and thrusts are determined for fixed supports. As a result of this study, the following conclusions are enumerated:

Offsetting the supports inwardly compensates for the thrust at the base, and outwardly increases restraining moments at the top of the vertical member.

The economy in the use of rigid frame bents, particularly of large span, is partly offset by the increased cost of foundations.

The thrusts are particularly important in one story bents, as for two story bents the loads from the second tier compensate in part for the thrusts resulting from the loads on the first tier, and following the different signs of the thrusts from the 1st and 2nd tiers, it is useless to attempt to compute the total thrust for the case of variable loads.

The length of the offset is a function of the ratio between the stiffness of the beam and that of the legs and the span of the loaded beam.

For a uniform load, the stiffness of the horizontal and vertical members being equal, the inward offset distance must be 1/12 of the span in order fully to compensate for the thrust. The positive moment at the center of the beam is, however, increased approximately 10 per cent.

It is impossible to set up an equation for the magnitude of offset fully to compensate for the thrust in case of moving loads.

Offsetting the supports outwardly increases the negative moment at the joint and reduces the moments in the beam, thus for an offset of $\frac{1}{16}$ of the span the moment of

the joints for uniform loads is increased nearly 20 per cent. However, the thrust at the support is correspondingly increased.

With fixed supports, compensating for the thrusts reduces the restraining moments at the joints to such an extent as to increase very materially the participation of the beam in carrying the load.

In conclusion, the author states that the offset of free supports for one story single bent rigid frames may prove of value in artificially increasing the moments at the joints.

The fully restrained frame, however, does not lend itself to this particular treatment.

Stresses in concrete pavement slabs

M. G. SPANGLER (Associate Structural Engineer, Engineering Experiment Station, Iowa State College). Highway Research Board (Dec. 1935). Reviewed by VERNE W. McCOWN.

This paper is a preliminary report on a current research project being conducted at Iowa State College, to supply experimental evidence bearing upon some of the assumptions utilized by Clifford Older in his interpretation of the Bates Road Tests and by Dr. Westergaard in his analytical solutions for stresses in pavement slabs. Loads have been applied to experimental slabs at a corner and the tensile strains in the top surface of the slab have been measured by means of optical lever extensometers.

The strain measurements, together with observation of the shape of structural corner breaks, both in the field and the laboratory have led to a new hypothesis relative to the distribution of stress, or the locus of maximum moment in a slab when loaded at a corner. It appears that the locus of maximum moment is a curved line which may lie anywhere between a circular curve having the corner of the slab as a center and a line tangent to this curve at right angles to the corner bisector. Also, the moment is non-uniformly distributed along this path, being greater near the bisector than at the edges of the slab. The factors which control the variations in shape of the locus of maximum moment between the described limits are not discernible.

Since the greatest stress in the slab under this hypothesis will occur when the locus is a circular curve and the least stress will occur when it is a straight line normal to the bisector, analyses incorporating these limiting cases are introduced and two expressions, one for the maximum and the other for the minimum probable tensile stresses along the corner bisector are derived. These equations, when evaluated with constants applicable to the first experimental slab studied, define two stress curves, one of which is about 50 per cent greater than the other. In general the measured stresses in this slab lie between these limiting curves and are nearer the curve of maximum probable stress.

A comparison between both the measured and analytically determined stresses and those obtained by Dr. Westergaard's analysis reveals substantial agreement between them as to the distance from the corner at which maximum stresses occur. Also his conclusion that the magnitude of subgrade reaction has relatively little effect upon stresses is verified. The expression here derived for minimum probable stress coincides with his expression for maximum stress.

The studies reveal the need for further experimental evidence relative to the relationship between subgrade reactions and deflections and the distribution of deflections in the corner region of slabs.

Construction of the Canning Dam, Western Australia

RUSSELL JOHN DUMAS AND VICTOR CRANSTON MUNT, *Journal of the Institution of Engineers Australia*, Vol. 8, No. 1, Jan. 1936, p. 1. Reviewed by J. R. SHANK.

The Canning Dam, a concrete structure, is being constructed in Western Australia to supply water for the City of Perth. Ninety per cent of the employees are on a part time basis under the Western Australian Government's Employment Scheme "to provide useful employment for workers on sustenance." The dam is 218 ft. high and 1600 ft. long with a 20-ft. roadway on top. It is estimated to cost $5,250,000. The average cost for all grades of concrete to date is $9.56 total.

The concrete is being made from a cement manufactured at Riverside Australia from shells and clay, a yellow natural sand,and a crusher run granite 2½ in. maximum size with a fineness modulus of 7.75. The proportions are 1:1.57:4.32 and 1:2.09:5.75 which produce compressive strengths at 28 days of approximately 3200 and 2600 p. s. i. respectively. The slump varies from 1 to 4 in. averaging about 3 in. No attempt is being made to hold the water cement ratio to any particular figure though the records indicated that it has been held between 0.88 and 0.94.

The cement is being handled in bulk. Special steel boxes 7 x 7 x 4 ft. are used in the transportation; two on a small freight car from the mill and one on a truck at the site. The transfer is made by means of a gantry crane. The saving due to this bulk handling is 90½ cents per cu. yd. of concrete or about $355,000 total.

The transportation of the concrete is being accomplished by the use of towers, chutes, and counterweight balanced trusses having as much as a 90 ft. radius. The chutes and trusses are carried on a cable way. The cost of the entire transportation plant is 10½ cents per cu. yd. The working grades of the chute lines are between 1½ to 1 and 2¼ to 1. Grades steeper than 2 to 1 are rarely used and then usually only at the tower. Concretes having as low as 1-in. slumps have been effectively chuted on 2.2 to 1 slopes.

Vertical construction joints are being constructed every 45 ft. down to a depth of 80 ft. and every 90 ft. below that. An 8-in. diameter bitumen water seal and copper water stop are used in each joint; the water seal at a concrete dowel projection and the water stop at a straight part between. The bow of the copper water stop is maintained by means of an asbestos-cement cover plate cemented to the copper by means of bitumen. The units for pouring are blocks 6 ft. high containing from 240 to 320 cu. yds. They are so constructed that any water which may tend to follow the vertical or horizontal joints will be compelled to change direction through 90 degrees a large number of times before appearing on the down-stream face. Inspection galleries are being constructed near the top and near the bottom, with 8-in. drainage ducts spaced 5 ft. connecting them. The foundation was good solid granite. It was impossible to force any grout into this material.

High elastic limit steel as reinforcement for sustained loading tests on slender concrete beams

H. J. GILKEY AND G. C. ERNST (Iowa State College). Highway Research Board (Dec. 1935). Reviewed by VERNE W. McCOWN.

This report constitutes Chapter IV of a continuing cooperative project between the Highway Research Board and the Iowa Engineering Experiment Station conducted under the general auspices of the project committee. The project is made up of a series of investigations of problems relating to the use of high elastic limit steel as reinforcement for concrete, with special reference to questions bearing upon possible increased design stresses for this material. Chapters I-III were published in Proceed-

ings Highway Research Board, Vol. 14, Part I (1935), p. 255-314. The scope of these chapters was as follows:

Chapt. I. Questions and Their Status. (p. 258-270).

Chapt. II. References with Brief Summaries. (p. 271-283).

Chapt. III. Design Procedure and Possible Economies from the Use of Higher Design Stresses. (p. 283-314).

For this report (Chap. IV) primary tests were made on 22 beams, 3 by 6 in. by 11 ft., tested under sustained static loading in dry air after 28 days of moist curing, followed by two or five months in dry laboratory air prior to application of load. All except four beams were reinforced with ¼ in. deformed rail steel bars and constitute the main series. Two beams (that broke after a few weeks under load) were unreinforced, and two others were reinforced with ⅜ in. deformed rail steel bars. Design steel stresses ranged from 15,000 to 46,000 p. s. i. and concrete stresses from 700 to 1,920 p. s. i. Design bond and diagonal tension stresses were below 70 and 50 p. s. i. respectively. Steel ratios varied between 0.30 and 1.21 per cent, with three beams reinforced for compression with steel equal to that in tension. Applied loads at each third point of 10-ft. spans were 140 lb. on the plain beams and from 185 to 700 lb. on reinforced beams. Center deflections and strains on steel and concrete were measured. Auxiliary weight and shrinkage data were taken, and crack surveys were made. The concrete offered considerable tensile resistance, even at the high stress combinations, and measured strains and deflections are below the normal computed values. Elastic deflection is a function of the stress in both steel and concrete, but flow-shrinkage deflection seems to be practically independent of steel stress and directly proportional to the concrete stress. Shrinkage is important, tending to lower the tensile steel stress and to increase compressive strains and beam deflections. Warping from unsymmetrical placement of reinforcement was one of the shrinkage effects noted on auxiliary beams. For the slender beams used in these tests, there was no indication of hazard or of unusual or unsatisfactory behavior within the maximum range of stress employed. Deflections were not excessive, and cracks were not wide, although there is little present basis for defining the limiting width of crack that constitutes a corrosion hazard.

Aggregate grading in relation to concrete mix design

H. N. WALSH, Bul., Inst. of Civil Engrs. of Ireland, Vol. 62, No. 6, April 1936, p. 197-36.

Reviewed by J. C. PEARSON.

This paper, one of a series by the author on the general subject of aggregate grading and proportioning, is directly supplementary to his previous paper, "Aggregate Grading and Concrete Quality" (see Review, A. C. I. JOURNAL, Sept.-Oct., 1933), in which typical aggregate gradings for crushed stone and gravel aggregates with maximum sizes of ¾ and 1½ in. were derived. The present paper extends these studies to cover gradings of crushed stone aggregate with maximum sizes of 2 in. and 3 in. These grading curves are shown on six charts, one for each maximum size and type of aggregate, and each chart has 5 curves, corresponding to 10, 8, 6, 5 and 4 parts of mixed aggregate by volume to 112 lbs. of cement. The author explains that each of these curves represents the coarsest grading that is suitable to make dense and workable concrete in the proportions indicated. These charts are supplemented by four others, one for each maximum size aggregate, which give cement content, gal. per sack, and C/W by weight, plotted against the ratio of mixed aggregate to cement by volume.

These 10 charts are the basis of the author's method of design of concrete mixes, which he states can be approached equally well from any one of the following angles:

1. Arbitrary proportions.
2. Definite cement content per cu. yd.
3. Definite strength, with high density and good workability.
4. High unit weight, with medium strength and low shrinkage, as for mass concrete.

It is interesting to observe that the author presents no strength charts, on the ground that the curing, age, and type of cement are too uncertain to warrant the use of assumed values. He prefers to use the equation $S = K\ (c/w\ \ -.5)$, in which the strength, S, depends upon the cement-water ratio by weight, c/w, and a constant, K, depending on the three variables indicated. Thus K is to be determined by preliminary tests and established for the materials and conditions at the work.

The paper is divided into four parts, (I) showing how the gradings are derived and the effect of departures from them; (II) the application to concrete mix design; (III) Specification requirements and a discussion of them; and (IV) the use of the charts for vibrated concrete.

This paper is well worth study by those interested in the design of concrete mixes, indicating as it does, quite different procedure from that commonly followed in this country. We are fast getting away from the use of volume proportions, and we prefer to design on the basis of strength as indicated by some form of the cement-water relation, rather than pay close attention to the grading of the aggregates. In either case, however, the adopted mix will depend upon trial batches, and the final adjustment will involve more or less attention to the aggregate gradation. Whether we subscribe, or not, to Prof. Walsh's ideas of correct grading, we may well hold to the idea that grading is not of minor importance, that it should vary with class and type of concrete, and that there is still much to be learned about the subject.

Stress in concrete and reinforced concrete resulting from volume changes

F. Campus, University of Liege, *Bulletin No. 24.* Reviewed by R. L. Bertin.

This paper was written as a result of observations by the author and others of cracks in reinforced concrete structures, localized by the presence of reinforcing bars to which they are parallel. These cracks were noticed along the lateral ties of reinforced concrete piles, also along the reinforcing bars of beams, along the vertical stirrups of deep beams and the reinforcing rods of slabs and walls.

The author attributes the formation of these cracks to the restraint offered by the bars to the shrinkage of the concrete, setting up internal tensile stresses in the concrete in the immediate vicinity of the bars in a transverse direction.

The internal tension set up in concrete in the direction of the reinforcing bars received considerable attention. This stress is modified by the restraint offered by the adhesion or friction between the concrete and the bars, whereas tension due to transverse shrinkage is not so modified and therefore is of greater magnitude than the former.

The author evaluates mathematically the stresses induced in a plane of a circular disk by the uniform shrinkage of a surrounding medium of unlimited dimension and deduces therefrom the tension in the medium in the vicinity of the disk.

Thus for concrete, assuming a moderate shrinkage of 4×10^{-4}, a tensile stress in the order of 280 p. s. i. can exist in the concrete at the periphery of the embedded bar which diminishes rapidly as the distance from the point of contact increases.

He extends the mathematical analysis to include the evaluation of stresses produced by a cylindrical bar, concentrically embedded in a cylinder of concrete, also in a sphere of concrete surrounding a concentrically located sphere of specific and constant volume, and finally the internal stresses in a cylindrical spirally reinforced concrete column under the effect of shrinkage. From these studies and observations, the author draws many conclusions:

(1) The pressure on reinforcing bars due to the lateral shrinkage of concrete increases with the thickness of the encasing concrete but tend rapidly toward a limiting value.

(2) The maximum tension in the concrete at the points of contact with the bars does not vary much with the thickness of the embedding concrete, and is approximately equal to that which results from the total restraint of the shrinkage.

(3) That the tensions decrease rapidly as the distance from the bars increases and are minimum at the free surface of the embedding concrete and therefore are the weaker the greater the thickness of concrete covering.

(4) The maximum internal stresses induced in concrete by the restraining effect of the reinforcing bars are considerably greater in the transverse direction than they are longitudinally but are localized in the vicinity of the bars.

In general, the internal stresses are governed by the relative volume of the heterogeneous elements in the medium. In concretes and mortars, the element with varying volume is the hydraulic binder, the aggregate particles being practically of fixed volumes. Even without reinforcing steel, mortars and concretes are subject to considerable internal stresses. This is evident from the fact that concretes and mortars shrink less than pure cement pastes. Therefore, the less the shrinkage of concrete as compared with that of pastes, the greater the internal stresses.

This explains why fines in concrete increase the tensile resistance while they reduce that in compression. As a result of the study of spirally reinforced columns, the author deduces as follows:

(1) The shrinkage of concrete induces transverse stresses in the column.

(2) That the spiral reinforcement is compressed, thus reducing its effectiveness within the elastic period under working load.

(3) That the core is subjected to an appreciable tension in all its mass which tends to increase its resistance in compression under load.

(4) That the shell is subjected to tangential tensions which are increased by the swelling of the core and the stretching of the spiral when the column is under load which explains the premature falling off of the shell under high loads.

In general, shrinkage provokes in spirally reinforced columns very complicated internal stresses having certain unfavorable characteristics which restrict the useful field of spiral reinforcement.

The carbonation of unhydrated portland cement

D. G. R. BONNELL, *Building Research Paper No. 19*, Department of Scientific and Industrial Research (London), May 1936. SUMMARY BY AUTHOR.

When portland cement is stored in such a manner that air has access to it, changes take place in the cement, which, in most cases, have a deleterious effect upon its properties. The nature and extent of this deterioration and the causes of the changes by which it is brought about are questions of importance to manufacturers and users alike. Although these problems have attracted the attention of research workers during the last 50 years, knowledge of the phenomena concerned is still far from complete.

In the course of investigations relating to the practical question of the deterioration of cements in storage, at the Building Research Station a few years ago, it became apparent that a study of the effects of exposing cements to the action of carbon dioxide might well produce results from which fundamental facts might be deduced. The study has been pursued, the degree of aeration being extended beyond the point at which a cement becomes useless for all practical purposes. Results indicate a new and promising line of attack upon several problems connected with the constitution of cement and the behaviour of the various constituent compounds.

A study of the specific effects of exposing three normal and one rapid-hardening portland cements to moist air and carbon dioxide has shown that, while moist air free from carbon dioxide has little effect, and dry carbon dioxide none, mixtures of the two decompose certain constituents of cement and have the effect of rapidly increasing the loss on ignition and the carbonate content with increasing time of exposure. The increase in the water content with time is very small, but definite. The amount of carbon dioxide absorbed during exposure to an atmosphere of moist air and carbon dioxide is directly proportional to the time of exposure until a certain critical carbon dioxide content is attained. Beyond this point, the rate of absorption is less and remains constant. The same applies also to the increase in the loss on ignition with exposure. This critical point is of significance in that, while being constant for any one cement, it varies with different cements.

There seems to be no doubt that this critical point is that at which a definite amount of carbonate is formed, and that this amount bears no relation to the free lime content of a cement.

Of considerable importance, of course, is the effect of aeration upon the tensile strength of a cement. Experiments over a wide range of carbon dioxide contents showed that the four cements studied behaved very much alike.

The investigation has shown that the actual effect of a definite amount of carbonation depends upon the amount that had previously occurred. With a fresh cement, comparatively small amounts may seriously affect the early strength without in any way reducing the 28-day strength, which remains almost unchanged until a considerable amount of carbonation has taken place, after which it decreases rapidly with further aeration. It is probable that the main reaction occurring during early aeration is the decomposition of the tricalcium aluminate content of a cement, while during the later stages it is the tricalcium silicate content which is mainly affected. From this it follows that, for early ages, the tricalcium aluminate content exercises a predominant influence on the strengths of both neat cement and mortar specimens, while, at later ages, this constituent has little or no influence and, in fact, may reduce strength.

In a study of the effect of aeration on the setting times of the cements, it was found that the results obtained varied very widely and were not quantitatively reproducible.

A phenomenon of peculiar interest which was observed is that the behaviour of aerated cements as regards the development of tensile strength differs in neat cement and in mortar specimens. Initially, with mortars, increasing carbonation has no effect on strength, while with neat cements a reduction in strength occurs. Mortar strengths also persist to a higher degree of aeration than is the case with neat strengths i.e., mortars develop strength after a cement has lost power to develop strength in the neat state. Indeed, a cement aerated to the degree that no tensile strength at all was developed in neat briquettes at 7 days gave strengths in 1:3 sand mortar of 105 p. s. i. at 1 day, and 209 p. s. i. at 7 days. This remarkable result was not due to the high proportion of mixing water required for mortars as compared with neat cements, but

appears to be intimately associated with the size and concentration of the aggregate particles.

These observations suggested a series of experiments the results of which form the basis of a hypothesis to account for the behaviour of cement and aggregate in a mortar.

Developments in concrete making—M. E. Freyssinet's new process

M. E. FREYSSINET (read by T. G. Gueritte at a joint meeting of the Institution of Structural Engineers and the Société des Ingéniéurs Civils de France, March 1936). Published in abstract *Concrete and Constructional Engineering* (England), April 1936, p. 209. The review following is from an editorial in the same publication, p. 207.

The objects of the paper were (1) to describe the properties and uses of the material to which the author has given the name "treated concrete"; (2) to show the advantages to be gained by initial tensioning of the reinforcement; and (3) to outline the theory of capillary solids developed by M. Freyssinet.

The principles involved in the manufacture of "treated concrete" are the use of the lowest possible water-cement-ratio and high proportions of fine and very fine aggregates, and vibration followed by compression to eliminate the water films. The results are concrete having a compressive strength of 14,000 p. s. i. two or three hours after the set begins when aluminous cement is used, and similar results in 12 to 24 hours with portland cements. Moreover, if vibrated and compressed concretes are heated to 212 deg. F., a strength of more than 5000 p. s. i. can be obtained in less than two hours in concrete made with portland cement, and ultimate strengths of 17,000 to 21,000 p. s. i. are reached.

The application of initial tension to the reinforcement to induce compression in the concrete is not new, but it has hitherto been found to be very difficult to carry out in practice. It is here that M. Freyssinet has made a notable advance in decreasing the deformation in the concrete and steel and preventing fissuring. To see a reinforced concrete pipe of 17 in. internal diameter and only $1\frac{3}{8}$ in. thick withstanding a test of about 1250 p. s. i. was a revelation to those present at the meeting. The pipe, we understand, had only been made six hours before the test. To the eye its surface bore little resemblance to an ordinary concrete pipe; in appearance it was more like a glazed stoneware.

The remarkable economy obtained by M. Freyssinet's method of construction is illustrated by the description of an I-girder 65 ft. 6 in. long, 2 ft. 9 in. to 3 ft. 10 in. deep, with a 6-in. web. The girder weighs 150 lb. per foot run. With a total test load of 940 lb. per foot the reinforcement is stressed to 78,000 p. s. i. and the concrete to 2056 p. s. i. in compression. The measured deflection was equivalent to that in a similar and similarly-loaded steel girder with stresses of 7100 p. s. i. Although submitted to alternate application and release of the test load no signs of fatigue appeared. It is estimated that a similar girder with a span of 327 ft. and an average depth of one-twentieth of the span would carry a useful load of 300 tons, and would have a factor of safety of two on the steel and four on the concrete.

At the Marine Station, Havre, where the buildings were settling at the rate of $\frac{7}{16}$ in. to $\frac{7}{8}$ in. a month, the new method of construction was successfully applied to transmit the loads to a firm stratum 65 ft. below the toe-level of the original piles and approximately 100 ft. below ground level. Between the footings of each row of transverse columns lightly reinforced mass concrete was placed so as to obtain a continuous beam from side to side of the building. In these beams a general compression was induced by steel tie-rods stressed to between 75,000 and 85,000 p. s. i.

Between the columns, circular vertical openings with horizontal grooves were boxed out of the new concrete, and through each opening a pile was sunk by hydraulic jacks. The piles are hollow cylinders with external and internal diameters of 24 in. and 15 in. respectively, and were manufactured above ground by a continuous process. Although the reinforcement in each pile is only about 7 lb. of steel per foot run, the piles are made to carry 300 tons and have a bending moment of 160 ft.-tons. During the manufacture of the piles the stress induced in the longitudinal bars was close to the elastic limit. The cement used was a special brand for work which is required to set in sea water, and which generally gives a low crushing strength. Notwithstanding the compression induced by pre-stressing the steel and the pressure required to jack down the piles it was found possible to sink them while the concrete was less than eight hours old. In four working days a length of 100 ft. of pile was manufactured and sunk.

Such are the properties of the new concrete. Its remarkable strength at early ages is the result of the application of five methods of obtaining high-strength concrete or expediting the hardening process, namely low water-cement-ratio, vibration, compression, heating, and tensioning the steel reinforcement. It is too early yet to attempt to prophesy the future of the process, but any improvement in the manufacture of concrete that is sponsored by the engineer responsible for the Plougastel bridge must receive the serious attention of all interested in concrete work. It is at any rate clear that the increase in working stresses from 600 p. s. i. to 1200 p. s. i. recommended by the Code of Practice is but a small step towards the stresses which may ultimately be used under certain conditions.

The Use of high-grade materials in reinforced concrete—a critical study of safety factors and allowable working stresses

P. Abeles, *Beton und Eisen*, April 20 and May 5, 1935.
Reviewed by William Quentin and B. Moreell.

The author adopts as his thesis, the view that current practice in reinforced concrete design is too conservative. After an exhaustive review of the subject, he summarizes his conclusions as follows:

1. Recent investigations have invariably shown that sections of reinforced concrete, designed by conventional methods, have a greater degree of safety than anticipated in the design; the amount of the excess depends largely on the steel ratio.

2. The ratio of calculated stresses at failure to those at yield point does not constitute a criterion of safety as regards stresses in the steel; the relation between stresses at yield point and working stresses varies with the diameter of the steel.

3. If a definite factor of safety for certain materials is stipulated, preliminary tests must be made to determine the corresponding steel ratio. The present conventional methods of calculation will give results appreciably different from those evolved in the tests, depending on the quality of the concrete and on the steel ratio.

4. The conventional design neglects tensile strength in concrete, employs $n = 15$ and assumes that with the appearance of cracks, the cooperation of the tension zone of the concrete is eliminated. It is the opinion of many authorities that the tension zone of the concrete is active in tension up to the moment of failure regardless of cracks.

The approximative methods of Steuermann and Gebauer, though devoid of strict theoretical foundations, check well with experimental data. These methods provide for the use of partial tensile cooperation of the concrete to the extent of ⅛ to 1/12 of the laboratory test strength.

5. As regards safety against cracks, Emperger states that danger arises only from cracks which expose the reinforcing steel permanently. Rigorous tests have proved that cracks of 0.3 mm width, permanently open, are not dangerous in dense concrete.

Danger resulting from cracks must not be judged from conditions prevailing at the time when fine cracks appear. Dangerous cracks appear soon after hair cracks when the reinforcement is small, and very much later for larger steel ratios.

6. Fine hair cracks, not only those due to shrinkage are unavoidable—and harmless —even in structures designed and erected by the best authorities. The denser the concrete the safer the reinforcement against corrosion; concrete of small density is subject to weathering and allows its reinforcement to rust without showing any cracks whatever.

7. An increase in tensile strength of concrete defers cracks. Wide cracks are less likely to appear in a concrete having greater capacity of undergoing deformation; manufacturers have not yet succeeded in improving this capacity.

8. It is considered very probable that in the near future a concrete of greater tensile strength will be produced. The compressive strength of concrete for structures subjected to bending requires no improvement. Relatively high tensile strength and lower compressive strength enhances the property of toughness.

9. It appears to be futile to employ a factor of safety by limiting the working stresses in the reinforcing steel to a certain value and to stipulate that high tensile steel should be used in combination with high strength concrete. A far more practical method is to stipulate a definite safety factor against rupture and against wide cracks.

10. A safety factor of 2 based on rupture is considered sufficient for ordinary structures, using a dense, uniform concrete and applying careful supervision during erection. If special protection against meteorological influences must be provided, a safety factor of 2½ will be sufficient, and a factor of 3½ must be used when harmful chemical influences are anticipated.

11. It appears that the use of high grade steel is economically advantageous. The design should then avoid high compressive stresses in the concrete, which would develop higher tensile stresses in the concrete, enhancing the danger of cracks.

12. According to Emperger, efficient bonding between steel and concrete is requisite to avoid cracks. Bond should therefore be improved by using deformed steel. High grade steel, having about twice the strength of the ordinary steel now in use, does not cost twice as much as the latter and will still be economically advantageous, if its cost be increased by processes of deformation in the factory.

13. As a disadvantage of high grade steel as discussed in this article (strength 8000 kg/cm) appears its brittleness, increasing the labor of bending and making welding prohibitive.

14. High grade steel in combination with concrete having the property of greater plasticity than it has now would be a fitting substitute for structures that under present conditions are built in steel.

15. To clarify fully the matter under discussion it is desirable (a) that a concrete be produced allowing a greater unit deformation in tension, (b) that more investigations be made to determine under varying conditions the degree of safety or the lack of it due to cracks, (c) that a certain correlation of steel ratios, tensile strength of concrete and the various grades of steel be established, on the basis of which safety of a structure may be gaged for any possible combination.

The cracks of reinforced concrete

HENRY LOSSIER, *Le Genie Civil*, Vol. CVIII, No. 8, Feb. 22, 1936, p. 182-186 and *Le Genie Civil*, No. 9, Feb. 29, 1936, p. 202-206. Reviewed by R. L. BERTIN.

The purpose of this article is to bring out the importance to be attached to the cracking of reinforced concrete. Although cracks always are an indication that the tensile resistance of the concrete has been exceeded, the cracks are not always dangerous. In some cases, they follow the adaptation of the structure to some particular condition and, outside of the question of appearance, they are to be viewed with alarm only when they are likely to affect either the resistance of the structure, protection of the reinforcement or the required tightness of the structure.

Under the heading "Formation of Cracks," the author points out that the development of cracks is progressive. Measurements indicate a gradual dislocation of the concrete which in cases increases its porosity though cracks do not appear. The reinforcing in concrete does not alter its property of rupturing when the elongation exceeds from .1 to .2 mm. per meter but it tends to distribute the stress, thus giving the concrete an apparent ductility so well demonstrated by the tests of Considere.

Cracks are subdivided into five classes, namely: (a) Cracks of resistance; (b) Cracks of conservation; (c) Cracks affecting tightness; (d) Cracks of adaptation; (e) Cracks due to shrinkage and temperature. Each class is dealt with in great detail.

Cracks of Resistance. Under this heading, the author discusses cracks due to excessive elongation of the steel with or without attending bond failure; cracks due to bond failure where the steel stress is low, and cracks due to shear. Much space is devoted to a correction of the old conception that vertical stirrups assist the concrete in resisting horizontal shear. The author points out that the so called shear cracks are in reality tension cracks. When these cracks occur, the tendency is to transfer the resistance to shearing forces onto the reinforcement.

Cracks of Conservation. The author distinguishes between cracks in which the bond is not broken, in which case the section of bar exposed is smaller than the width of the crack and those where the bond is broken, in which case the exposed part of the bar may exceed the width of crack. The importance of these cracks on the permanence of the structure is dependent upon its exposure to active elements, but in general if the former are less than two-thirds of a millimeter, no danger exists. The latter type of crack may affect the permanence of the structure. They generally appear in regions where the steel tensile stress is very high or where the bond stress is excessive. These conditions are generally found over the supports of continuous members where the amount of steel was insufficient. The effect of such cracks on the permanence of structures is apparently less than from the porosity of the concrete, particularly when the structure is exposed to active agents.

Cracks of Adaptation. Cracks frequently occur when the assumptions of design are not realized in actual performance. Continuous structures, monolithic arches insufficiently reinforced at the supports, hinged members lacking freedom of action, or structures founded on yielding soils are examples of such cases. It is a general law applying to hyperstatic structures that the weak elements tend to unload themselves automatically to the deteriment of the more rigid ones. Such cracks are not necessarily injurious if the stronger rigid members are capable of assuming the burden.

Cracks From Shrinkage and Temperature. The author claims that the real knowledge regarding shrinkage of concrete is still rudimentary. He discusses the physical phenomena of the shrinkage of plastic pastes, the action of humidity, plastic flow under stress, chemical reactions, etc., investigated by Le Chatelier, Chapman, Faber, Freyssinet, Pigeaud, and Giertz Hedstrom. He deals with shrinkage from two stand-

points; the uniform shrinkage of the whole mass and the local shrinkage affected by restraints, such as reinforcement.

In the second part of his paper, the author discusses suggested means of preventing the formation of cracks. He first discusses the limitation of the theoretical elongation of concrete in tension. By analysis and test, he shows that even without stressing the embedded reinforcement, the resistance of concrete in tension may be exceeded resulting from shrinkage, particularly if the reinforcement is not concentric with the concrete section in which it is embedded. He then takes up the use of special reinforcement designed to increase bond resistance. In this respect he points out that deformations on the surface of the bars do not increase the adhesion but add anchorage which tends to reduce the formation of cracks or to decrease their magnitude Attention is called to tests conducted on Isteg bars and others such as the Radix bar where the exposed length of bars in the vicinity of cracks were less than for smooth bars even when the unit tensile stress was greater. He warns, however, against the use of steels the yield point of which is raised artificially as compared with those having normal high yield point, the former being unstable under the action of heat, vibration or shocks. He discusses the influence of percentage of steel spacing of bars, general arrangement of bars on the formation of cracks, more as a matter of interest than of practical importance.

The effect of prestressing the reinforcement is discussed, reference being made to the work of Considere, Freyssinet, Baticle, Lund and Koenen, Dischinger, Von Emperger, Ruml, regarding application and methods of prestressing the reinforcement in such structures as bow strung members, ordinary beams, pipes, etc. The use of rust proof steels is given consideration as well as the selection of mixtures and cements producing concretes having low shrinkage values and higher ductility.

In conclusion, the author states that in general, cracks do not endanger the structures except in cases where they occur in regions subjected to tension which are not provided with reinforcement of sufficient strength to resist the stresses without assistance from the concrete.

He finally forecasts as the ultimate solution of the problem, the production of a concrete which will possess greater ductility and reduced shrinkage or possibly expansion. Such a product is now in the state of development and promising tests are under way which will be reported in the near future.

Discussion of a Report of Committee 609:

"Recommendations for Placing Concrete by Vibration"*

CONVENTION DISCUSSION

Raymond E. Davis (Professor of Civil Engineering, Univ. of California, Berkeley): In listening to the report of the Committee, certain items have impressed me as needing emphasis. One pertains to high frequencies. In this report, frequencies of 5,000 r. p. m. or higher are recommended. Job experience leads me to believe that frequencies as high as 10,000 would be desirable. I have in mind one job on an inverted siphon which pretty well illustrates the efficiency of high frequency over low frequency. The forms for the siphon were open on the outside at about half its height, and it was necessary to work the concrete through closely spaced reinforcement, from the side opening down into the invert. With a vibrator for which the frequency was approximately 3,500 r. p. m., it was found impossible anywhere near to fill the invert of the form. But when there was brought to the job a vibrator having a frequency of 7,000 r. p. m., the invert was filled with no difficulty whatever.

One other point with which I would disagree has to do with the vibration of concrete in layers below the one which is being placed. It appears that re-vibration in partially set concrete actually improves the strength of the concrete. In a series of tests on approximately 300—6 by 12-in. cylinders, the concrete was re-vibrated at several intervals after initial placement. This interval was varied from 5 minutes to 2 hours. Several cements were used, some of which took their initial set in less than 2 hours. The results of these tests consistently indicated that the longer the time that had elapsed before re-vibration the greater was the strength of the concrete. The maximum increase in the 28-day compressive strength due to re-vibration was about 20 per cent, and this maximum increase was about the same for one cement as for another.

*Journal, Amer. Concrete Inst., March-April 1936; *Proceedings*, Vol. 32, p. 445.

In one of the Government laboratories tests have been made recently to determine the effect of vibration of reinforcement steel during the setting period upon the bond strength. For these tests the reinforcing bars, some of which were horizontal and some of which were vertical, were vibrated at intervals of 15 minutes, until the concrete was 10 hours old. The results of the tests indicated that the 28-day bond strength was consistently increased by vibrating the steel until the concrete had reached the age of 7½ hours, when the bond strength was 75 per cent higher than for specimens for which the steel had not been subjected to repeated vibrations.

I am in favor of vibration as a means of compacting concrete, but not as a means of transporting it from one place to another in the forms; and I am convinced that on a good many jobs where wet mixes are employed over-vibration produces such a marked segregation as to damage the concrete. The use of vibrators in high slump concrete and the use of vibrators as a means of making concrete flow for considerable distances after depositing it in the forms should not be permitted.

Recently on one job where the concrete was made to flow by vibration over a considerable distance in the forms, an analysis of the fresh concrete was made. At a point distant 14 ft. from the place of deposit, it was determined that the cement content of the mix was 1½ sacks per cu. yd. greater than that at the place of deposit and that there was present in the mix only a small percentage of coarse aggregate above the ¾ in. size. Even though the concrete at this point contained 1½ more sacks of cement per cu. yd., its compressive strength was 20 per cent lower than that of the concrete at the place of deposit. The manufacturers of concrete vibrators are not backward in making claims which will promote the use of their equipment, but I have yet to hear a manufacturer claim that vibrators should be used for transporting concrete in the forms.

Air bubbles which appear on the surfaces of formed concrete appear to be particularly numerous when vibration is employed. Probably the principal reason for this is that stiffer mixes are used from which the air cannot readily escape. It appears that for the stiffer mixes, air bubbles may be considerably reduced by placing the concrete in thin layers.

L. W. Walter (Erie Railroad Co., Jersey City, N. J.): I think most of us feel we have had so little experience with vibrators that we are not qualified to make very many positive statements, so I shall try to express opinions. To my notion one feature was omitted in the

committee's conclusions—that the advantages from the use of vibrators are first in improving quality by using dry mixes; thus, with a given amount of material, using a lower water-cement ratio, improving quality through the use of a lower water-cement ratio. On the other hand, the economic advantage is in using harsher mixes to increase the amount of aggregate per unit of cement, thereby reducing the cement factor. I have been told, and I am inclined to accept the view as a sound one, that by increasing yield per unit volume of cement, a concrete has less tendency toward volume changes. If that be true, that vibration tends to reduce the volume changes, then the advantage is not merely one of economy, but of quality improvement.

Commenting on Mr. Davis' remarks about the vibrators being used to induce the flow of concrete within the form, I suspect that where a vibrator is utilized to facilitate the flow any considerable distance from the point of deposit, resulting concrete will be sloppy as in years gone by where a great deal of really detrimental segregation occurred through the flowing action after the concrete entered the forms. The flowing action of the concrete within the forms tends to separate the finer and lighter materials and carry them to the further end of the structure where they are dumped and cannot get any farther. The damage to the concrete resulting from this extended flow within the forms depends to a considerable extent upon the cleanness of the aggregate. If aggregates are moderately dirty, the tendency is for the dirt, clay or finer materials, whether sawdust shavings or any light material, or materials in a finely divided state, to gain speed on the coarser and heavier materials and the result is that a considerable portion of these materials are incorporated in a relatively small section of the structure. The water-cement ratio in the concrete at the far end of the flow is found to be considerably higher than that before the flow was induced.

Walter H. Wheeler (Minneapolis): When I began designing and building concrete as a construction engineer, I found that it paid to use external vibration on the column forms and wall forms of a job. I carried that practice on into the contracting business and for the last 24 years have carried it on in my professional work as an engineer. That was external vibration. Three years ago on a large job in Baltimore, a contractor was placing 730 cu. yd. of concrete in a building per day in a continuous run of 13 to 14 hours. It required a great many men spading and poling the concrete. Of his own volition, after he had poured three floors, the contractor adopted the internal vibration method. He was able to get along with half as many men on the floor as he had before, using one vibrator and he almost entirely

eliminated patching. Since that time I have made vibration a part of my specifications on all important building work. Last summer we used it on the Minneapolis Armory which had about 9,000 cu. yd. of concrete in it. The first floor slab was 60,000 sq. ft. in extent, 12 in. thick and was finished as placed. When you attempt to finish a 12-in. slab within a certain interval, it must be thoroughly consolidated, otherwise when you get through with your finishing operation you will have depressions and hollows. The slab was covered immediately after it was finished with burlap and sand, while the steel work and masonry work went on in the building for three months and then the cover was removed. After the cover was taken off, the floor was checked. It was found that the finished surface did not vary more than ⅛ in. from exact grade. The office part of the building has four floors and a roof, which are mainly flat slab construction. Plywood forms were used and it was the intention not to plaster or paint the ceilings. A very smooth finish was important. Vibration was used on these floors also, and except for the fins between the plywood panels and except for the stains which are hard to avoid, it would not have been necessary to touch these slabs in order to have a finished ceiling. Because of these two factors surfaces were given a very light rubbing with carborundum, which finished them satisfactorily. The other part of the story is that the concrete was designed for 2,500 p. s. i. with a fairly dry mix—1 cement: 2 sand: 3¼ coarse aggregate—not to exceed 6½ gal. of water. These solid slabs were vibrated, using 5½ gal. of water (most of the concrete had a slump of about two in., never over four in.) and the test cylinders from these slabs showed in all cases except one, a strength at 28 days of more than 4,000 p. s. i. The one cylinder under 4,000 lb., was above 3,900 lb. On that entire area of slab (about 100,000 sq. ft.) not a patch was put on after the forms were removed. We were unable to securs as good results on the concrete joist construction. Using the same concrete specification the cylinders averaged about 1000 p. s. i. less.

On columns and high walls I am not convinced that internal vibration is the entire solution. It is my experience that on such parts of the work you have got to have something in addition to internal vibration, you must use either poling and spading, as has been the custom in addition to the vibration, or you have got to use both external and internal vibration to get satisfactory results.

A. M. Philleo (U. S. Engineer, Zanesville, O.): On this matter of frequency, in competitive trials of vibrators, we have found that 4,500 is more efficient with large aggregates than a higher frequency; in fact, in competition, one vibrator that had a rated frequency of 10,000

did not begin to do the job that 4,500 did. However, in Class A concrete, with small aggregates, it does seem that quality goes up with the frequency.

A. W. Mall (Mall Tool Company, Chicago—convention remarks amplified by letter): There are many reasons to believe that higher amplitude or high speed of the vibrator is desirable, but for practical applications it is better to use a slower speed of 3,500 r. p. m. When a vibrating machine is applied to the job of compacting and placing concrete, there is considerable material to be moved or work to be done. The vibrator must be so designed that it will put the motor behind it to work and agitate or vibrate the concrete so that it will compact and fill out the forms solidly.

A concrete vibrator must be designed so that it will kick or move the material within a radius of 18 to 36 in. Speed alone will not accomplish this. It is possible to put into concrete a vibrator revolving at 20,000 r. p. m. and find that it hardly makes any impression on the material. On the other hand, placing another properly designed vibrator running at 3,500 r. p. m. into the same mass, the concrete is well vibrated and placed. Practically all vibrators on the market today use the same principle of revolving an out-of-balance weight on anti-friction bearings inside of a metal protecting housing which is inserted in the concrete. The energy of the motor is transmitted to the job of compacting the concrete through the out-of-balance weight which, in turn, is pounded through the anti-friction bearings into the housing of the vibrator and then into the concrete. The energy with which the concrete is vibrated depends upon the radius of the weight revolved, its specific gravity and the speed of the motor.

Considering the fact that all this motor energy must be pounded through the bearings into the concrete, it is obvious that if this work can be accomplished at speeds of 3,500 r. p. m., it is much more practical than a speed higher than this which would tend to destroy the bearings sooner and cause other mechanical failures in the equipment which would shorten the life of the machine. The reason many people believe that high revolutions are necessary is because with a given vibrator revolved at higher speed, more energy is used to vibrate the concrete, the power required to revolve the vibrator at the higher speed increasing as the square. In test, it was shown that a given amount of concrete could be compacted in a shorter time when the vibrator was speeded up. These same results could be obtained by using a vibrator operating at 3,500 r. p. m. which was larger and in which a heavier weight was revolved at a given speed of 3,500 r. p. m. The motor naturally would be larger in horsepower and therefore the

energy transmitted into the application of vibrating or compacting the concrete would be greater and therefore less time would be taken to compact a given mass of concrete.

In all our tests where the vibrator is properly designed, it boils down to the energy applied to the job of vibrating or shaking the concrete mass, the amount of time required depending upon the amount of motor energy applied to the particular job. It is sometimes necessary to revolve a vibrator higher than 3,500 r. p. m. where a limited space of 2 in. or less is available for the insertion of the vibrator; also, to reduce the weight of the machine carried about by the operator. A small vibrator revolving at high speed weighs less for a given amount of applied vibrator energy.

Vibrators can be made easily to revolve at 10,000 r. p. m. However, it can be assumed that the vibrator will have a life of only one-third of that of one which is running at 3,500 r. p. m. We have made numerous tests in which we measure the amount of motor energy going into the concrete at various vibration speeds. We have found that it is necessary that the weight revolved inside of the vibrator housing be of high specific gravity in order that the proper results be obtained. As a result of our numerous tests, we think it not necessary to revolve the vibrator at higher speed than 3,500 r. p. m. for 90 per cent of all applications. Probably on 10 per cent of the construction jobs it is advisable to run the vibrator at higher speeds because of the limited space of 2 in. or less in which the vibrator can be inserted.

Discussion of the Report of Committee 501:

"Building Regulations for Reinforced Concrete"*

There was preliminary discussion of parts of the new Code at the 31st Annual Convention, New York, and further discussion before, at and since the 32nd Annual Convention, Chicago, Feb. 1936, at which the new regulations, as there revised, were adopted as a tentative standard of the Institute. Insofar as discussion led more or less directly to revision it is not published and that portion of the discussion here presented is from the viewpoint of further desirable revision of the Tentative Code—Editor

STRESS IN FLEXURE NEAR A SUPPORT

John R. Nichols (Boston, by letter): The Building Regulations tentatively adopted by the Institute in February 1936, provides in Section 305 for an extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames, higher than at other sections in bending. I question the propriety of such a higher stress and the justification for its use.

Possibly I am in error in supposing this to be a survival of the ancient days when negative bending was ignored in the belief, or at least the hope, that such neglect was on the side of safety. Motive for continuing the practice is found in the considerable economy to be obtained by allowing a comparatively small excess in stress just for a short distance next supports. But this motive will hardly take the place of reasoning, if we are honest with ourselves, in any attempt to devise a comprehensive method of design that provides a uniform factor of safety at all sections and in all kinds of stress. The Building Regulations should aim to be no less.

The use of a higher stress adjacent to supports is sometimes defended on two grounds, (1) that the practice over a period of years has not resulted in failure, and (2) that the excess in stress exists only for a short distance.

The first of these defenses, if the fact is sustained by inquiry, might be offered to justify a higher compressive stress generally in bending

*Journal, Amer. Concrete Inst., March-April 1936; *Proceedings*, Vol. 32, p. 407.

but will hardly, by itself, warrant picking out sections close to supports for exceptional strength.

The second defense deserves more attention. Sections where fiber stress changes rapidly with distance from the section of maximum stress are sections of high shear, high diagonal tension. That is precisely what shear measures—rate of change of bending. Is there ground for believing that sections of high shear may safely bear excessive compression? I know of none. I am aware that in a cross-section the points most remote from the neutral axis and hence most highly stressed in compression are points subject to minimum unit shear approaching zero, and that as the unit shear on the cross-section increases toward the neutral axis the unit compression decreases. Nevertheless, as between a section of zero shear and a section of high shear, there appears to be no reason for conceding greater strength in compressive fiber stress to the latter. On the contrary here, if anywhere, are to be found diagonal tension cracks which should be expected to produce weakness rather than strength.

As for the quality of the concrete in the bottom of beams near supports here is less assurance of excellence than almost anywhere else. Here, if anywhere, will be found sawdust, chips, dried concrete and debris splashed from the filling of columns. Reinforcing bars for negative bending and for diagonal tension furnish maximum interference with the placing and consolidation of this concrete. The full depth of the beam from the tensile steel is less likely to be maintained; the bars are apt to be trodden down. The concrete is fully exposed to lateral distortion or to accidental injury. It seems to have not one redeeming feature in its favor.

Can I then have overlooked some important consideration which justifies high flexural compressive stress? That, of course, is entirely possible and the possibility is the reason for this discussion. Let the justification be brought forth and placed on record; or let us take such other action as may be consistent with our convictions. There is no place in engineering for wishful thinking.

SHEAR AND DIAGONAL TENSION

Paul Norton (*Maginnis and Walsh, Architects, Boston*): I hope I may be forgiven for being the voice of somewhat vehement dissent on Chapter 8, "Shear and Diagonal Tension." They say it is a difference of opinion that makes a horse race, and although present-day opinion hardly seems to attach to the construction industry an economic importance equal to that of racing, possibly an expression of disagreement may also be useful to the work of a committee con-

cerned with concrete construction. I do not altogether like this chapter. It seems to me unfortunate in that it emphasizes to such a degree formulas and the mechanics of computation, rather than the basic principles and physical conceptions that underlie the formulas. Instead of beginning by prescribing that the beam must be satisfactory for the shear requirements, and then telling how it may be made so, this chapter begins with an algebraic formula by which we may compute the amount of shear existing in the beam; it tells what the computer must do, instead of what the beam must do.

Then the title does not quite fairly represent the contents of the chapter; however, I would not have this corrected by changing the title; it would seem better to keep the text and change the sermon. Diagonal tension seems to me a very important part of this subject, and it is mentioned only in the caption—no direct reference to it anywhere else. May I try to be constructive by submitting in brief outline what it seems to me should be the principal content of this chapter? It ought to begin with the statement that the member must, in some way, be made capable of resisting the diagonal tension in the web. Then I would like to have it set forth that, for purposes of design, at least, the physical conception shall be entertained that the diagonal tension is directed at an angle of forty-five degrees to the axis of the beam, and that its numerical measure is the intensity of the shear. And then it might prescribe a limit to the intensity of shear that is allowable in a beam that has no web reinforcement, and again a limit for that in a beam with web reinforcement. None of these statements is in the chapter, and they seem to me the most important ones of all to be made. Then I think another physical conception should be presented for the purpose of designing web reinforcement; that the web reinforcing bars, no matter in what direction they occur in the web, are to be conceived as affording for resistance to diagonal tension the components of their stresses which have the direction of that diagonal tension. Now if these things are said, Formulas 14, 15, and 16A will be inescapable; all the formulas will be inescapable, except Formula 16, and I shall lack enthusiasm for that formula until someone gives me a physical conception that it can be tied to.

Professor Cross remarked yesterday† that the way of progress in reinforced concrete design lies in giving the designer more latitude. In the home town we have been trying to do exactly that in the formulation of a new building code. And we have found that the way to do it is to say less about how the engineer shall act, and more about

†"Why Continuous Frames," JOURNAL, Amer. Concrete Inst., March-April 1935; *Proceedings*, Vol. 31, p. 358.

how the structure shall act; to say more about the design and less about the designer. It seems to me that the distinction will run through the formulation of all parts of a building code or of any code of practice. It applies to the chapter on Bond as it does to Shear; I think it may apply to other chapters. I am interested in progress in these things, and the point of view seems to me of real importance. Thank you very much.

FLAT SLABS

Walter H. Wheeler (*Designing and Consulting Engineer, Minneapolis, by letter*): The Building Code of the American Concrete Institute is rapidly becoming the law of the land for reinforced concrete design. It is therefore important that this code shall be in all respects fair and impartial and based upon facts and experience as far as possible. Where experience or facts are lacking tests should be made as promptly as possible to develop the facts.

Sec. 1001, paragraph C. This paragraph is being interpreted to mean that flat slab buildings cannot be built 2 panels wide. It should be made clear that such is not the intention. The writer has designed many buildings and some bridges with spans ranging up to 49 feet, all or part of which are two panels wide. Some of these structures have been tested, some have been in service as long as 27 years. Neither the tests nor the service have shown any indication of weakness in these structures. On the contrary tests have shown that a flat slab two panels wide is just as rigid as a structure three panels wide. In the designs the moments in the reinforcing strips running across the width of the structure were adjusted in the same manner that the moments in continuous beams or slabs would be adjusted for a similar condition.

Similarly the writer has designed numerous flat slab structures with very irregular panels applying the common principles of balanced moments. Both tests and experience have shown that the results are entirely satisfactory.

In this paragraph and elsewhere in the code special emphasis is laid on bending stresses in columns in flat slab construction. As a matter of fact both the interior and wall columns in a structure having girders running in one direction supporting continuous slabs running in the opposite direction are subjected to greater bending stresses due to unbalanced loads than they are in a flat slab structure. In the one way slab structure the entire unbalanced load acts to transmit bending stresses to the column in one direction; whereas in the flat slab structure the load and consequently the moments travel to the column from all directions. The bending on the column is therefore divided into

segments acting in different directions, thus reducing the maximum bending moment in the column.

The definition of a column capital as given in the code reads "An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab." I propose that a sentence be added to this definition reading as follows:

"A framework of structural steel adequate for the same purpose." Steel column capitals are being used. Tests have shown them to be equally as effective as the standard concrete capital. It will promote progress in the art to recognize steel capitals in the code.

The formula for determining slab thickness as given in the code for slabs without drop panels is obsolete. As a method of approximating slab thickness for preliminary design purposes it may serve. It results in excessively thick slabs. There is ample test data to prove that to be true. The code has set up the unit shear and fiber stresses to be used as the basis of design.

The basis is defined for determining negative moments over the columns and shear around the column caps, from which the theoretical unit stresses can be determined. There is ample test data to show that the theoretical unit fiber stresses are not approached in the actual structures. Experienced designers do not follow this rule of thumb method for determining flat slab thickness. Such a method is not used for beam and slab or joist construction. Why put something in the code which is being constantly misinterpreted, and which penalizes such a useful construction as flat slab.

I refer to published test data as follows:

1. Deere and Webber Building, Minneapolis, Minn.: 4-way flat slab; panels 19 ft. 1 in. x 18 ft. 8 in.; slab 9 $\frac{3}{16}$ in. thick; no drop panel; live load design 225 lb. per sq. ft.; test load 350 lb. per sq. ft.; maximum concrete stress in outer fiber due to negative moment at the column 800 p. s. i.; maximum steel stress 24,200 p. s. i.

2. Curtis Leger Building, Chicago, Ill.: 4-way flat slab; panel 17 ft. 10 in. x 19 ft. 0 in.; slab 8 in. thick; no drop panel; design live load 200 lb. per sq. ft.; test load 500 lb. per sq. ft.; maximum concrete stress in outer fiber due to negative moment at the column 930 p. s. i.; maximum steel stress 11,500 p. s. i.

3. U. S. Appraisers Stores Building, Baltimore, Md.: 2-way flat slab; panels 23 ft. 11 in. x 23 ft. 9 in. adjacent panels 13 ft. 8 in. x 23 ft. 9 in. One panel tested where slab is two spans wide and one panel where slab is five spans wide; slab 13½ in. thick; no concrete capital or drop panel, steel capital enclosed within depth of slab; design live plus superimposed dead load 275 lb. per sq. ft.; test load over 500 lb. per sq. ft.; maximum concrete stress in outer fiber due to negative moment at the column 510 p. s. i.; maximum steel stress 7,000 p. s. i.; designed according to U. S. Navy Code.

It will be noted that by the formula for slab thickness in the proposed code the Deere and Webber slab would have to be 10½ in. thick or 1$\frac{5}{16}$ in. thicker than it is. The Curtis Leger slab would have to be 9.7 in. thick or 1.7 in. thicker than it is. The U. S. Appraisers slab would have to be 14½ in thick or 1 in. thicker than it is.

The water cement ratios set up in Table 302 (a) result in concretes that run 1,000 p. s. i., or more higher at 28 days than given in the table assuming that a good workable mix containing fine to coarse aggregate in the ratio of 2 to 3½ is used.

Discussion of a paper by Carl A. Menzel:

"Studies of High Pressure Steam Curing of Concrete Slabs and Beams"*

CONVENTION DISCUSSION

A. L. Philleo (U. S. Engineers, Zanesville, O.): I should like to ask what per cent of silica is used as a substitute or admixture?

Mr. Menzel: Some aggregates provide a portion of the silica needed, and for very lean mixes the aggregate often provides all the silica needed, but in the very rich mixes there might be a substantial deficiency. In that case you will have to supply it by adding fine silica—of about the same fineness as the cement. The amount of silica to be added in any one case can be reasonably determined by trial, and this method has been outlined in the second paper in the JOURNAL of the Institute in connection with the hollow concrete block studies (Sept.-Oct. 1935). We give a procedure there for determining the amount of silica for any type of aggregate and mix.

J. H. Chubb (Penn-Dixie Cement Corp., New York City): What is the cost of the chamber that you would use—say 6 ft. in diameter and 46 ft. long?

Mr. Menzel: I have the approximate cost figures on a complete curing equipment consisting of a cylinder about 6 ft. in diameter and 60 ft. long. That will require about a 40 h. p. boiler, and, with the auxiliary equipment, such as pumps, etc., the entire equipment can be installed for about $10,000, including the small building required to house the cylinder. That cylinder would be completely insulated to provide for a minimum cost for fuel. Such equipment (6 by 60 ft. cylinder) can cure approximately 1,000 8 x 8 x 16-in. units at a time and under some circumstances as much as 500 cu. ft. of cast stone. Cinder or Haydite units with a 12-hr. curing cycle, would permit steaming 2,000 blocks per day. Assuming a 200 day-a-year operation, you could steam, with such equipment, approximately 400,000 units per year.

*JOURNAL, Amer. Concrete Inst., May-June 1936; *Proceedings*, Vol. 32, p. 621.

Chairman Pearson: I understand the quantity of steam and the actual value of that steam, if you got it from another source, would not be very great as compared to the cost of the overhead and the engineer and the shell and everything else.

Mr. Menzel: That is right; I am glad you brought up that point; if you are so favorably situated that you can buy your steam from some other source, you can reduce the cost of this steam curing equipment to about $7,500 and you can about cut curing costs in half. With a 12-hr. cycle, the cost of steaming cinder or Haydite units would be about 1½ cents, but if you can buy your steam at from 50 to 85 cents per 1,000 lb, steam curing costs are just about ¾ cents per unit.

With cast stone, the cost for steam curing would be ordinarily about 4 to 8 cents per cu. ft., but if you can buy your steam and eliminate the expense of the licensed engineer and fireman and other things, you can reduce the cost for cast stone to about 2½ to 5 cents per cu. ft. Now these costs are based on intensive operations of the steam curing equipment. If the equipment lies idle, costs would increase. The costs I have mentioned are estimated, based on an assumption that the cost of the equipment, $10,000, is being written off in a 10-year period, at an interest rate of 6 per cent. Some maintenance is also included. If some old sand-lime-brick equipment is used, costs would be lower in proportion to the lower investment charges.

Earl A. Soloman (Penn-Dixie Cement Corp., New York City): You say you get no benefit above 350° F.

Mr. Menzel: No benefit in time, it would take approximately as long to complete the curing, to obtain the same strength. We tried 400°F. which is nearly double the pressure. 350°F. corresponds to 120-lb. gage pressure, while only 50° higher corresponds to 200-lb. pressure. While we find no additional benefit from higher pressures, our tests indicate that by lowering the temperature to 3000° steaming for 40 hours instead of 6 would be necessary to get the same strength.

Mr. Soloman: Are those results available?

Mr. Menzel: We did not think them important enough to publish. We are trying to keep the pressure down. If block made today can be used 24 hours later—block which are dry, white in color and low volume change and low susceptibility to leaching and efflorescence, I think that is as much as can be expected from curing. I think you should not go to these higher pressures.

Mr. Chubb: What is the moisture content of the product after it comes out of the curing chamber, compared with the same product cured in the air for a week?

Mr. Menzel: That depends somewhat on the type of aggregate; if you have sand and gravel units, you can lose about two-thirds or more of the moisture present in the block at the time of molding; they come out comparatively dry and lose very little more moisture when exposed to the air such as we have in the laboratory, which is comparatively dry air. Cinders and Haydite and the more porous aggregates have a higher moisture content; they would lose a little more moisture later but not very much. The units could actually be installed in the wall as soon as removed from the steam chamber.

George T. Dieckmann (Northwestern States Portland Cement Co., Mason City, Ia.): What mixture was used in the 8 x 8 x 16-in. block?

Mr. Menzel: We had mixtures which we varied from lean to rich. I can give it to you in terms of cement content per block if that would mean anything to you. We had a cement content from a little less than three pounds per unit to about four and a half pounds per unit—from 20 to 35 block per sack of cement.

Mr. Dieckmann: If with high pressure steam curing you can get 35 blocks from a bag of cement as compared with 18 blocks with ordinary curing that would be a consideration.

Chairman Pearson: I should like to cite our experience. Some years ago we were making studies along this line and we had no difficulty in getting 47 block per sack of cement. We aimed at two pounds of cement per block and got a block in 24 hours which passed specifications without difficulty.

Benjamin Wilk (Manager, Standard Building Products Co., Detroit, Mich.): I would caution any one who attempts to make 47 blocks out of a bag of cement; our experience is that it is not a question of curing but of being able to put the block into the curing room. We can make, from the standpoint of strength, much leaner mixes than we use, but we do not, primarily because we cannot handle satisfactorily a block of the 47 to-the-bag type, especially when we are trying to make a light block. Today the tendency is to get low weight, and with low weight go thin webs, and with thin webs the tendency is toward more cracking as you handle the blocks. It is not a question at all in my mind of the strength you get from the curing process, it is entirely a manufacturing point—to be able to put the block satisfactorily into the kiln.

Has any study been made of the difference in volume change between the block made this way by high pressure steam curing, and blocks, say, that are cured 28 days?

Mr. Menzel: We did not make these studies directly on the concrete block, but we had established thoroughly from the other specimens and from the preliminary studies, that the volume change of concrete can be reduced from 50 to 75 per cent by curing under pressure at 350°F.

D. R. Collins (Portland Cement Association, Chicago): I believe this higher pressure steam curing is already getting practical application. I talked to a manufacturer three weeks ago who said he had closed an order that day for more than 500,000 units. His selling argument was based on high pressure steam curing, and he assured me that was only part of the orders he had received.

B. E. Brevik (Portland Cement Association, Wauwatosa, Wisc.): Has any investigation been made as to absorption in high pressure steam cured products?

Mr. Menzel: There was an appreciable difference between steam cured concrete and moist cured concrete. Steam cured concrete does not appear to be wetted near as easily as moist cured material; in fact, the surface seems for a while to be water-repellant. The absorption total would probably be about the same for the pressure cured block as for the moist cured material under certain conditions, but the rate at which water is absorbed is apparently much lower with pressure cured blocks.

Mr. Wilk: In your analysis of cost, Mr. Menzel, you said it was based on intensive production and that would mean two cycles a day. In making lighter weight units, using the more porous aggregates, would it be possible, from a commercial standpoint, to have two cycles a day.

Mr. Menzel: Yes, it would. That was a 12-hr. cycle; we allow 3 hr. for gradual heating and 8 hr. for constant temperature and pressure and ½ hr. for pressure release; that makes 11½ hr., allowing another half hour for emptying the cylinder and filling it up again. The cylinder would be warm, so while the units were being rolled into the cylinder they would be gaining temperature, and as soon as they get in, the heating period would begin. You can actually get two cycles per day with porous aggregates. With dense aggregates—sand and gravel made usually of rich, dense concrete for exterior use—we believe that an 18-hr. cycle would be far superior to 12 hr. because there should be a safeguard against very fine surface checks; you cannot see them, but our tests with the solid slabs indicate that very probably they are there. They probably would not do much harm in interior use, but for exterior use we want to have the very finest concrete we can get to stand the weather.

W. A. Jennings (*Southwestern Portland Cement Co., Osborn, O.*): You get differences in color from different positions in the steam cylinder, do you not?

Mr. Menzel: I think there is some danger of that, if you do not provide baffle plates which will deflect your condensation to the side of the cylinder. We were able to install a curved plate, in the upper half of the cylinder, about a half inch from the shell, and most of the condensation was deflected by this curved plate to the side of the cylinder. In that way we eliminated the possible stain from rusting. On the other hand, in a commercial products plant using pressure steam curing, I noticed no stain; I rather expected it, but we did not find it.

M. H. McKenzie (*Corn Products Refining Co., Chicago*): What was the fineness of the silica as compared with the fineness of the cement used as a substitution? Second, did I understand you to say that your tests showed that you could replace cement with silica and secure equivalent strength? If that is so, how high a percentage of substitution could be made?

Mr. Menzel: The silica which is most effective has about the same fineness as cement. We went thoroughly into the question of the fineness of the silica. We made studies from sizes as small as five microns or less, and studied the effect of the progressive increases in particle size up to No. 48 sieve. We found we got the best results with a silica that was of about the same fineness as the cement. Now it is possible to get strength equal to or greater with silica mixtures than with ordinary cement when the aggregate contains no siliceous particles. That is not quite true in the case of excessively lean mixes, as we sometimes have them in concrete masonry units, but with rich dense concrete, such as might be used in cast stone, it would be desirable to use silica and in that case you might have to use 30 to 40 per cent of fine silica with the cement.

Mr. McKenzie: Did you determine any comparative costs between this silica and cement?

Mr. Menzel: Silica varies in price in different parts of the country, but I think that the silica price will not be a big factor; it would probably be about the same as the cement or slightly higher.

Mr. Dieckmann: Which silica do you recommend?

Mr. Menzel: I believe you will get the best results and be on the safest side if you use a silica such as they have in Ottawa—ground silica sand. There are differences in silica, but I think they are not very great. We found similar reactions with silica bearing material,

but I am just a little afraid of using them when we can have available so readily the ground silica from silica sand.

J. Clifford Evans (Nazareth Cement Co., Easton, Pa.): What is the difference between steam curing at 350° and simple heating in air to 350°. Does the steam supply moisture to the block?

Mr. Menzel: The curing of concrete depends on three essentials; time, favorable temperatures and the continued presence of moisture. In steam curing we accelerate the chemical reaction between water and cement and we have an auxiliary reaction between the lime liberated between the hydration and the hydrolysis of the cement and the silica present. Now moisture is an essential for all curing of concrete. If you should take concrete and expose it to hot air at 350°F. you would have some very disappointing results. You use the saturated steam to insure that the concrete will be cured in the continued presence of moisture. I think that is the difference between steam curing and oven curing, as you might call it. You are sure that the concrete will not be robbed of the moisture so necessary to complete the chemical reaction.

Discussion of a paper by R. E. Copeland and C. C. Carlson:

"Tests of the Resistance of Concrete Masonry Walls to the Penetration of Rain"*

CONVENTION DISCUSSION

H. Vanderwerp (Medusa Portland Cement Co., Cleveland, O.): I think Mr. Copeland's paper is very timely. During these depression years there has not been much construction, and we all confidently believe that there is a considerable deferred demand in the offing. The extent to which concrete will share in this construction work will depend, of course on the service it can render. The objections to concrete have largely been that it is a drab looking and porous material. Now if we can, by painting the surfaces, meet these objections and render the walls relatively impervious to moisture and give them color, probably we could increase the demand for concrete. That being the case, it would seem that it is extremely important to do this properly. A few years ago when stucco construction was coming into vogue, we had many kinds of stuccos. A number of them failed. We now have many kinds of cold water paint having various properties. Would it not be properly the function of the American Concrete Institute to devise a specification for a portland cement paint, or a paint, let us say, and a method for applying it, that would give these results that Mr. Copeland's paper indicates are possible? I should like to offer that suggestion.

George P. Dieckmann (Northwestern States Portland Cement Co., Mason City, Ia.): Mr. Copeland's paper makes me feel awfully bad, if we have to admit that we have to paint our concrete masonry. There is no reason, with proper mortar joints and proper mortar, why a concrete block basement wall should not be dry. You could not paint a basement from the outside. In the city of Minneapolis, which is one of the greatest consumers of concrete masonry, it would be

*Journal, Amer. Concrete Inst., March-April 1936; *Proceedings*, Vol. 32, p. 485.

practically impossible to put this thing over. We have got to make concrete products which are water tight and which are durable. If we have to admit that we have to paint them, why, we decrease the consumption of concrete blocks.

Mr. Copeland: Basement walls usually are not so exposed to rain and it is general practice in the construction of concrete basement walls either to plaster or cope the outside. This particular investigation was conducted with reference to walls above grade and which are exposed to rain. As s result of our tests, which were made on a large range of types of units, mortars, etc., it was our conclusion that, under severe rainstorm conditions which we simulated, concrete masonry walls will leak. I think that it to the advantage of concrete products manufacturers to sell a type of construction which they can guarantee will not leak unless the surface is made watertight with stucco, paint, or other effective coating. Undoubtedly there are many examples which products manufacturers can point to that have proven entirely successful and have never leaked. There are also many jobs which have leaked, which we have never heard about and which are acting to retard and restrict the sale of concrete masonry because people tell their neighbors about it and the neighbors tell their neighbors and concrete is handicapped in that respect. Painting is not an expensive treatment, it is extremely effective, it has advantages from an architectural standpoint. We must not assume that other types of masonry units or walls constructed of those units do not leak, because we know that they do leak. The problem is not confined to concrete masonry.

Frank Muenzer (*Multiplex Concrete Machine Co., Elmore, O.*): I have been experimenting in the last four years on block made with an oscillated face. With a glass tube sealed into an oscillator block, I subjected it to ten pounds water pressure for 36 hours, and no water penetrated to the back of the block.

Mr. Copeland: That is very interesting, and our results show there was some beneficial effect from the oscillation of the unit. However there is a difference between the effect of hydrostatic head on a masonry wall and the combined action of wind and rain.

D. E. Parsons (*National Bureau of Standards, Washington, D. C.*): It appears that the Portland Cement Association has undertaken an investigation similar to one started at the Bureau of Standards about nine months ago. The work of the Bureau was sponsored by and is being supported by four of the governmental agencies. The Bureau

is making tests of walls of brick masonry, walls with brick masonry facing with a back of clay tile or concrete units and walls with stucco facings. The tests thus far made have included exposure to capillary action and exposure to water with air pressure. The method of applying the pressure is somewhat different from that used by Mr. Copeland and his associates, but the effect should be nearly the same. We adopted the scheme of applying a static pressure to the exposed face, a pressure equal to five pounds per square foot, which is somewhat greater than that being obtained in the tests by Mr. Copeland. That method was chosen so that, if the opportunity offered, we would be in position to repeat the test in the next year or two years or five years hence, after having exposed the masonry to weathering conditions. The results thus far obtained indicate a wide divergence of behavior of the masonry walls under exposure to wind and rain. Dampness penetrated some walls in three minutes or less; others remained dry during exposures of more than one week. The results indicate conclusively that the most important factors are workmanship, the way in which the joints are filled. The differences in the properties of bricks and the properties of mortar are relatively insignificant in comparison with the effect of the workmanship. Like Mr. Copeland and his associates, we have found that stucco facings provided excellent protection to the wall against wind and rain. The time period required for water to come through an eight inch wall with stucco face has varied from approximately one day to fourteen days in our tests. Thus far we have not made tests on walls of concrete masonry units; they are on the program and we hope to have the results in the next year.

A. S. Douglass (*Construction Engineer, Detroit Edison Co., Detroit*): I appreciate very greatly, as a user, the work being done; it is of tremendous practical value. However I feel that it may be too subtle to be solved by the laboratory type of research, and I feel that a suggestion directed toward initiating a different type of research may be of value. Mr. Parsons suggested the feasibility of conducting such a research as I have in mind, namely, a long time field record, followed by observations, putting competent observers on to actual masonry work, recording carefully and minutely the type of workmanship, the ingredients and proportions; all those details which are so numerous, and then from year to year and for many years thereafter observing performance. I am convinced of the necessity of that sort of research from having observed initmately for some years, an attempt to solve this problem practically in the field. I think this matter of seepage correlates itself closely with the matter of efflorescence in the masonry.

In one building, at a corner we observed utterly different performance; same brick, same mortar, same brick mason, same weather, same locality; on one face we had efflorescence and on the other we had no efflorescence. Why? Of course the weather exposure is obviously the answer, but what is the difference? Where is the dividing line? Certainly that demonstration proved that the whole solution does not lie in the brick, mortar or workmanship, all of them in that case being identical. I am sure that others who have watched the matter of efflorescence carefully, (I regard efflorescence as a part of or very closely allied to this problem) have doubtless noticed that under window sills of cast stone or natural stone you will get heavy efflorescence, while in pilasters running the height of the building between windows there will be none. Again you have many factors which are identical. Why the difference? Under some window sills you won't get it and under others you will. One thing we have tried seems to correlate with other observations. We have inserted under coping stones (I am referring to the design now so prevalent—the corniceless design so much used today—no overhang, no protection for the parapet masonry) a copper strip protruding about an inch and a half and we have found that very effective. In a large building we have inserted such a strip and adjacent to it on the same exposure, left a stretch where we have not inserted this strip, and the benefit of that inch and a half umbrella is very decided. I should like to repeat the suggestion that some organizations such as the Bureau of Standards or the Portland Cement Association initiate over a long period such a field follow up observation. I should like to add also that it is my conviction, corroborating what has just been said in a different manner, that the careful tooling of masonry joints is one of the most important things that can be done.

Ben Moreell (Commander, C. E. C., U. S. Navy, Washington, D. C.): Mention of the work at the National Bureau of Standards brings my department into the picture. We are one of the four participating departments sponsoring these tests; in fact, the Navy Department initiated these tests, and the reason is that we made such a survey as Mr. Douglass suggested and found such a condition of affairs that it would be very little exaggeration to say that practically all masonry walls exposed to hard rain and wind leaked. Now what is the cause of that leakage? We feel that a good deal of the leakage occurs not because of the character of the masonry units but because of the poor workmanship. We found that it pays to flash openings and that is what we are doing. We found that one prolific source of leakage was the practice of supporting floor joists and floor beams on the wall

and decreasing the thickness of the masonry at those points without any attempt to waterproof the interior surface. It is difficult to get to that surface on a job; it costs money. We also found that tooling the joints would reduce leakage. As a result of our observations, we drew up a code of practice for the construction of masonry walls, which provides for a minimum cover of masonry over joists and floor beams, the flashing of openings, tooling of joints, a certain maximum thickness of joints, and it provides certain mortar compositions. Then we decided that the safest thing to do after taking all these precautions was to provide for draining the water from the inside of the building after the walls leaked, and that is what we are doing. And then, to provide an added factor of safety, we inaugurated this cooperative research at the National Bureau of Standards.

In connection with Mr. Copeland's paper, one point brought out might well stand some elaboration. I notice that he directed the flow of air at an angle of approximately 45 degrees, using a velocity of 25 m. p. h. Some of Mr. Copeland's statements left the impression that this represented severe exposure conditions. You gentlemen probably know that the usual practice, at least the usual Navy practice, in northern climates is to design for 30 lb. of wind pressure. It is usual for rain to accompany the wind. Now using the 25-mile wind that Mr. Copeland used, I think it figures 2½ lb. per sq. ft. If you apply the correction for the angle of incidence between the wind and the wall, it is reduced to something like 1¾ lb. per sq. ft., which is an extremely low pressure. We find that we have higher winds than that, a good deal higher; so we design our structures for higher pressures. In basing conclusions on the values of the various walls, I believe it would be prudent to bear in mind that they are relative values. At the National Bureau of Standards it is obviously impracticable to use a wind velocity that would give a pressure of 30 lb., and the probability is that such a high velocity is not needed; that would be about 88 m. p. h.; but we recognize that fact and our object in interpreting the tests from the Bureau of Standards will be to determine relative values rather than absolute values. We do not intend to say when a certain wall indicates no leakage under the conditions of the tests at the National Bureau, that that wall is going to be free from leakage in actual practice.

L. W. Walter (Erie Railroad Co., Jersey City, N. J.): A railroad freight terminal warehouse built several years ago in Ohio, 5-stories, concrete with brick curtain walls, fairly well constructed—developed leakage at the joints between the brick curtain walls and the concrete

columns and beams. We did not experience water coming in during the average thunderstorm, but when we got a rain of longer duration, such as we get with a northeaster, with wind of considerable driving force, the water came through in places and ran down on the floor. The leakage was considerable. Freight of a nature that moisture would damage, was sometimes stored in the building and our problem was to stop the flow of water at these joints only; we had no trouble with water coming through the curtain wall or elsewhere in the building other than the joints between the brick and the concrete. After trying to stop the water from the outside, I suggested the use of asphalt emulsion applied in a narrow ribbonlike strip along the joint at the line of contact between the brick and the concrete. We used the generally accepted best way of applying asphalt, which was to dampen the surface first so as to avoid the rapid evaporation of water from the emulsion, and we gave it in some places three coats, in some places four coats with pleasing results. In this particular building, we were not much concerned about the inside appearance. Had we been interested in the inside decorating, I would have suggested applying a coat of aluminum paint to the asphalt emulsion, and thereafter you could put on almost any sort of paint. However we did not do this, so I cannot recommend this unqualifiedly as a good method for stopping such leaking. In this one particular case it was a success.

Chairman J. C. Pearson (*Director of Research, Lehigh Portland Cement Co., Allentown, Pa.*): If I interpret Commander Moreell's remarks correctly, he was rather defending the laboratory type of study, even though Mr. Douglass perhaps intended to discount it somewhat. Perhaps as a laboratory man my sympathy is the other way, because as we grow older and get more experience, we are less inclined to look to the laboratory investigation to solve any problem finally—it does not make much difference whether we are developing a new product or trying to find a cure for some constructional fault, no matter what sort of investigation you have in the laboratory, you generally have to take it into either the plant or the field to get the final answer. The great object of the laboratory investigation is to get the leads at the least possible expense of time and money.

AUTHORS' CLOSURE

The authors thank the various discussers for their constructive comments concerning the report and the general problem of building watertight masonry structures.

In "high-lighting" the similar investigation now under way at the National Bureau of Standards, Mr. Parsons gave us something to look

forward to with great interest. We concur in his observation concerning the importance of workmanship. In our experience small channels or passages left between the masonry unit and mortar generally will result in copious leakage. These defects will occur with improper or careless workmanship regardless of the kind of materials employed.

Mr. Parsons raised the point of the difficulty of obtaining and preserving even air pressure or wind velocity. We were perhaps fortunate in experiencing very little difficulty on this score. During the investigation the velocities were checked frequently, readings being taken at four points in the cross sectional area of the duct about 12 in. from the wall face. Reasonably constant velocity conditions prevailed throughout the investigation.

The remarks by Mr. Douglass were timely and illustrate the complexity and, to a certain extent, indefiniteness of the problem. The development of a highly rain-resistant masonry wall construction is only one phase in obtaining masonry buildings which do not leak. Messrs. Douglass and Walter called attention to some of the details which must receive special consideration.

Both Commander Moreell and Mr. Pearson question the severity of the test exposure based on their belief that higher wind velocities frequently occur with rain.

The selection of the 25-mile wind velocity was based originally on limited data but since has been substantiated by an extensive survey of rain intensities and accompanying wind velocities conducted principally with respect to nine cities: Chicago, Cleveland, St. Louis, New York, Washington, D. C., Jacksonville and Miami, Florida, Galveston, Texas and New Orleans. These cities were chosen because of their geographical and climatic differences, population importance and probably completeness and length of climatological records. All data were extracted from records of the United States Weather Bureau and with the assistance of that Bureau. Length of periods studied varied from seven years for Cleveland to from 16 to 20 for the other cities.

The survey showed that of the 1,759 rains of one inch or more total precipitation recorded for these cities for the periods covered, all but 36 were accompanied by winds whose recorded 24-hour average velocity was less than 25 miles per hour. Considering these rains the test wind velocity is equal to or greater than the average velocities which accompanied 98 per cent of the rains and is somewhat lower in the case of 2 per cent of the rains.

Twenty-one of the 36 winds which averaged more than 25 miles per hour occurred at New York City and the average velocity of these 21 winds was 29 miles per hour which does not greatly exceed the velocity used in the test.

It was not practical to simulate the fluctuating velocity and changing direction of natural winds in the test procedure and a constant wind velocity continuously applied on the wall face was employed. The constant and continuous character of the test wind seemed to correspond more closely with the average 24-hour velocities of wind accompanying rain than to any other velocity value of such winds. Since maximum velocities frequently are sustained only for relatively short periods they were disregarded in favor of the average velocity values in determining the proper exposure to use.

The angle of incidence employed is believed to follow to a large extent actual conditions where the rain is directed at an angle much of the time due to the natural shifting of the wind and to the turbulence and directional interference caused by the surrounding structures, trees, shrubs and other obstructions.

Discussion of a paper by W. H. Herschel and E. A. Pisapia:

"Factors of Workability of Portland Cement Concrete"*

J. C. Sprague (U. S. Engineers, Huntington, W. Va., by letter): The paper presents valuable data on workability and, although not solving the problem, takes us one more step from that state of affairs described so aptly by Jackson and Kellermann[11] as follows:

> "In spite of strenuous efforts on the part of many investigators, no satisfactory laboratory test for workability has as yet been developed. Furthermore, this whole matter of workability is tied up so intimately with methods of handling and finishing used on the job that it is impossible to set up any laboratory standard which will give more than comparative results."

The authors have brought out the fact that the workability of concrete has a number of aspects. Harshness, shear resistance, adhesion and mortar loss tests have all been presented as measuring some facet of the workability of concrete. In the first paragraph of their paper, the authors have made it clear that the methods described were not given as fulfilling "all requisites of the needed test procedure or as yielding numerical values indicative of workability . . . "

Since that elusive property of concrete, workability, is very much in the limelight today, it is thought that a few remarks on the various aspects of workability as presented in the paper and in connection with a test developed by the writer, might prove of some interest.

Powers[12] has given a definition of workability as follows:

> Workability is that property of a plastic concrete mixture which determines the ease with which it can be placed and the degree to which it resists segregation.

Pearson[1] says:

> More attention should be devoted to the study of segregation, which is after all the final and necessary test of adequate workability . . . a complete analysis of the constituents of freshly placed concrete is not necessarily required for its solution; in fact there are various indirect methods which may be simpler and less subject to error.

*Journal, Amer. Concrete Inst., May-June 1936; *Proceedings*, Vol. 32, p. 641.
(11) "Studies of Paving Concrete," *Public Roads*, Vol. 12, No. 6, August 1932.
(12) "Studies of Workability of Concrete," by T. C. Powers, Journal, Amer. Concrete Inst., February 1932; *Proceedings*, Vol. 28, p. 419.
(1) References (1) to (10) will be found with the original paper.

Near the beginning of their paper, the authors stated that "any concrete of whatever richness or leanness, or howsoever proportioned, may have its water content adjusted to give any desired flow within reasonable limits . . . " About two years ago the writer had an experience which does not seem to agree with this. For a given gradation of coarse aggregate, the water content could be changed as much as 5 gal. per cu. yd. of concrete without affecting the slump or flow of the mixture. This condition obtained when the aggregate was poorly graded, and was explained by the fact that individual particles of coarse aggregate interfered with each other and set up an "arch action" which made it impossible for the concrete to slump or flow. As soon as the aggregate was graded properly, changes in flow followed changes in water content. The concrete which showed no signs of being mobile with changes in water content was definitely a harsh mixture, and a measure of harshness such as that proposed in the paper (Fig. 1) would probably have brought out the different degrees of harshness resulting from changes in gradation of the aggregate. At that time, however, variations in the harshness of the concrete were measured by water-gain, or bleeding action of the mixing water. It was observed that considerable water rose to the surface in the harsh mixture, whereas those concrete mixtures which were plastic and had cohesiveness (stickiness) showed comparatively little water-gain.

As a result of these observations, and of a need for evaluating quantities of various commercial fines to be used in lieu of natural fines lacking in concrete fine aggregate, a test procedure was developed whereby the bleeding action of mixing water from fresh concrete mixtures was measured. The mechanical principle of the so-called "water-gain" table, used in the test, is the same as that of the flow table; but instead of measuring the flow of the concrete, the water rising to the top of the mixture is extracted and weighed. The extracted water, divided by the weight of the original water content, is the percentage of bleeding.

It was found that the bleeding of mixing water varied over a wide range for different mixtures having the same degree of mobility. The mobility of the concrete mixtures was controlled by a remolding test, a modification of the test developed by Powers[12]. The slump test was used as a check on the remolding test. The results of bleeding tests for a few of the mixtures investigated are tabulated below:

Determination	Mix Designation							
	A	*B*	*C*	*D*	*E*	*F*	*G*	*H*
Remolding Effort...... (No. of Drops)	18	17	18	18	19	17	18	18
Slump—Inches........	2.8	2.9	3.0	3.0	3.0	2.9	3.0	3.3
Water-Cement Ratio...	0.90	0.90	0.93	0.95	1.05	1.10	1.08	1.20
Per Cent Bleeding at 120 Minutes........	13.2	8.6	13.8	13.1	14.3	11.4	13.6	18.7

Mixes *A* and *B* were for a natural sand and crushed limestone coarse aggregate; *C* and *D* represent concrete mixtures in which blended fine aggregates consisting of natural and crushed sand were used with limestone as a coarse aggregate; *E* and *F* were all limestone aggregate concretes; Mix *G* consisted of a sandstone fine aggregate and limestone coarse aggregate; and *H* was an all sandstone aggregate mixture. It will be noticed that the per cent bleeding ranged from 8.6 for Mix *B* to 18.7 for Mix *H;* and that the water-cement ratio was increased 0.30 to give equal mobility. It does not, however, follow from this that the increased water content was the cause of increased bleeding. In some cases the opposite trend occurs.

In other tests which were made to evaluate the effect of sub-sieve fines on bleeding action only in concrete mixtures, it was found that the per cent bleeding ranged from 1.7 to 10.0 per cent. In this case no attempt was made to control the mobility of the concrete.

While the tests referred to were confined largely to effect of fine aggregate, and of sub-sieve fines, on water-gain, it is tentatively suggested that a test which combines a measure of mobility and of bleeding action might serve as a measure of workability as defined by Powers[(12)] and Pearson[(1)]. It is an axiom as old as concrete itself that workability of concrete is a function of the gradation of its component parts. Further, it is logical to believe that particle interference and segregation in concrete are caused largely by poor gradation. If we continue, then, with a premises that water-gain in concrete results from segregation and particle interference, the natural inference is that bleeding of mixing water and a measure of mobility will measure two or more of the different facets of workability. These last two assumptions are unsupported by results of actual tests, although the writer now has a series of tests under way in this connection. It is believed that the phenomenon of bleeding is closely associated with workability, or rather with the property of cohesiveness (stickiness) which is one of the attributes of workability.

The bleeding tests is suggested primarily as a segregation test because the water is the one thing that separates most readily and obviously, and in *most instances* this is directly in line with workability. This is also quite in line with the well known properties of porous substances, in which permeability is more a function of pore size than percentage of pore volume. It seems quite reasonable that a relation does exist between the bleeding of concrete and the harshness and segregation in the mix, assuming of course that no more mixing water has been used than is necessary for the required mobility; certainly it would seem undesirable to use concrete which shows excessive bleeding because in such concrete, lack of homogeneity may result. Rather would it seem desirable that the mixture be such that it is just on the verge of bleeding, for when in this condition, just enough fines have been used to retain the water and not too much to require an excess of water because of their presence.

In this discussion it has not been attempted to analyze the data presented by the authors—suffice to say that a very definite effort has been made to find a measure of workability. It is hoped, however, that the comments contained herein may prove of interest, and that they may suggest a way to a simplified measure of the more important of the various aspects of workability.

AUTHORS' CLOSURE

Replying to comments of Mr. Sprague, it is known that in general, for a given mix, there is a decrease in flow with a decrease in water content. However, we have found and we believe it is fairly well known, that with very harsh mixes the flow can not be reduced below a certain value, as the flow increases with a further decrease in Water content. What is needed is a method of measuring wetness which (in contrast to the slump and flow tests) is not influenced by shear resistance. Such a wetness test might then be expected to show the effect of variations in water content, no matter how harsh or dry the mix.

Discussion of a paper by George A. Maney:

"ANALYSIS OF MULTIPLE SPAN RIGID FRAME BRIDGES BY THE SLOPE DEFLECTION METHOD"*

L. T. Wyly (*Structural Engineer, Meredosea, Ill., by letter*): This paper by the originator of the slope deflection method is characterized by completeness of treatment and simplicity of method. It should further advance that development of rigid frames of concrete which, according to E. J. Mehren,[1] has largely been stimulated in America by Professor Maney's original development of slope deflection in 1915[2] and somewhat later by Professor Cross' variant moment distribution method.[3] Elsewhere the writer has outlined the main steps in the evolution of this basic and flexible method of analysis.[4] He desires here to discuss briefly one or two assumptions made by the author in his paper.

Assumption No. 1—Moment of Inertia of Section. The assumption made by the author applies to slab structures. However, frequently use is made of T-beam or of slab-and-girder units and convenient assumptions are made as to the moment of inertia of the combined section. So far as the writer knows, no experimental evidence has yet been published on this question. More definite information on this matter is desirable.

Assumption No. 4—Span Lengths—The author has assumed clear spans in his analysis, demonstrating that this assumption gives results closer to those obtained by arch analysis than the center to center assumption gives. The writer desires to extend the slope deflection method a little to investigate this question more closely for the frames treated.

*JOURNAL, Amer. Concrete Inst., March-April 1936; *Proceedings*, Vol. 32, p. 495.

(1) "Concrete, Yesterday, Today and Tomorrow," E. J. Mehren, JOURNAL, Amer. Concrete Inst., March-April 1935; *Proceedings*, Vol. 31, p. 345.

(2) "Secondary Stresses and Other Problems in Rigid Frames—a New Method of Solution" by G. A. Maney, Bulletin No. 1, Engineering Studies, Univ. of Minnesota, March 1915.

(3) "Analysis of Continuous Frames by Distributing Fixed-end Moments," Hardy Cross, *Transactions*, Am. Soc. C. E., Vol. 96, p. 1.

(4) Discussion by L. T. Wyly on "Successive Elimination of Unknowns in the Slope-Deflection Method," John B. Wilbur. *Proceedings*, Am. Soc. C. E., August 1936, p. 921.

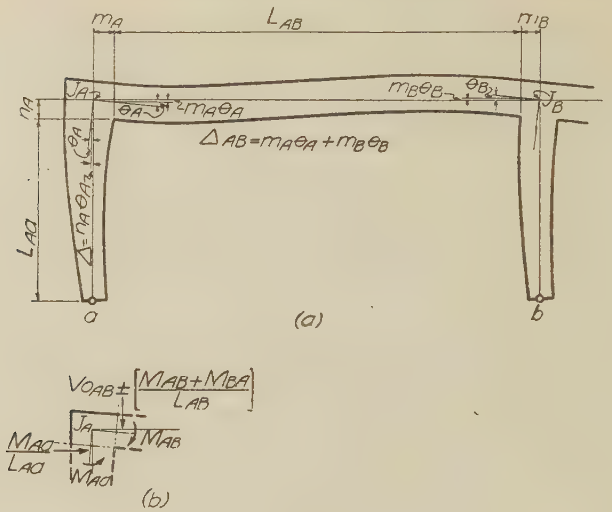


FIG. 1

SINGLE SPAN CASE – SYMMETRICAL Author's Fig. 3

Loading Due To Slab Plus 45#/□'

Formulae

$MJ_{AB} = M_{AB} + M_e + V_{oAB} m_A$

$MJ_{Aa} = M_{Aa}\left(1 + \frac{n_A}{L_{Aa}}\right)$

Let $\frac{K_{Aa}}{K_{AB}} = K_r$

$MJ_A = M_{AB} + M_{Aa}\left(1 + \frac{n_A}{L_{Aa}}\right) + M_e + V_{oAB} m_A$

$M_{AB} = M_{F'AB} - K_{AB}[C_{AB} - C'_{AB}]\theta_A$

$M_{Aa} = -K_{Aa} C''_{Aa}\left[1 + \frac{n_A}{144 L_{Aa}}\right]\theta_A$

$M_e = 0$

$V_{oAB} = \frac{1}{2}\Sigma W_x$

$MJ_A = M_{F'AB} + V_{oAB} m_A - \theta_A \left\{K_{AB}(C_{AB} - C'_{AB}) + K_{Aa}\left[1 + \frac{n_A}{144 L_{Aa}}\right]\left(1 + \frac{n_A}{L_{Aa}}\right) C''_{Aa}\right\}$

$$\frac{\theta_A}{K_{AB}} = \frac{M_{F'AB} + V_{oAB} m_A + M_e}{[C_{AB} - C'_{AB}] + K_r\left(1 + \frac{n_A}{144 L_{Aa}}\right)\left(1 + \frac{n_A}{L_{Aa}}\right) C''_{Aa}}$$

Example

$M_{F'AB} = 156100'^{\#}$

$V_{oAB} = 13535^{\#}$

$V_{oAB} m_A = 13535^{\#} \times 2.25 = 30454'^{\#}$

$\frac{n_A}{144 L_A} = .0010$

$\frac{n_A}{L_{Aa}} = .1475$

$C_{AB} = 9.6$ $K_r = 10$

$C'_{AB} = 7.4$

$C''_{Aa} = 7.4$

$$\frac{\theta_A}{K_{AB}} = \frac{156100 + 30454}{2.2 + 10(1.001)1.1475 + 7.4} = \frac{2140}{K_{AB}}$$

$M_{AB} = 156100 - 2.2[2140] = 151390'^{\#}$

$M_{Aa} = -10 \times 7.4[1.001]2140 = -158510$

$\left.\begin{array}{l} MJ_{AB} = 151390 + 30454 = 181844'^{\#} \\ MJ_{Aa} = -158510 \times 1.1475 = -181900 \end{array}\right\}$ Check

$M_c = 176200 - 151390 = 24810$

FIG. 2

TWO SPAN CASE – SYMMETRICAL Author's Fig. 12

Loading Due To Slab Plus 45 #/□'

Formulae

$$MJ_{AB} = M_{AB}\left(1+\frac{m_A}{L_{AB}}\right) + M_{BA}\frac{m_A}{L_{AB}} + V_{oAB}m_A + M_e$$

$$MJ_{Aa} = M_{Aa}\left(1+\frac{n_A}{L_{Aa}}\right)$$

$$MJ_A = M_{AB}\left(1+\frac{m_A}{L_{AB}}\right) + M_{BA}\frac{m_A}{L_{AB}} + M_{Aa}\left(1+\frac{n_A}{L_{Aa}}\right) + V_{oAB}m_A + M_e$$

$$M_{AB} = M_{F'AB} - K_{AB}\left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C_{AB}\right]\theta_A$$

$$M_{BA} = M_{F'BA} - K_{AB}\left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C'_{AB}\right]\theta_A$$

$$M_{Aa} = -K_{Aa}C''_{Aa}\left[1+\frac{n_A}{144L_{Aa}}\right]\theta_A$$

$$M_e = 0$$

$$V_{oAB} = \tfrac{1}{2}\Sigma Wx$$

$$MJ_A = M_{F'AB}\left(1+\frac{m_A}{L_{AB}}\right) + M_{F'BA}\frac{m_A}{L_{AB}} - \theta_A\left\{K_{AB}\left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C_{AB}\right]\left(1+\frac{m_A}{L_{AB}}\right) + \left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C'\frac{m_A}{L_{AB}}\right] + K_{Aa}\left[\left(1+\frac{n_A}{144L_{Aa}}\right)\left(1+\frac{n_A}{L_{Aa}}\right)C''_{Aa}\right\} + V_{oAB}m_A + M_e$$

$$\frac{\theta_A}{K_{AB}} = \frac{M_{F'AB}\left(1+\frac{m_A}{L_{AB}}\right) + M_{F'BA}\frac{m_A}{L_{AB}} + V_{oAB}m_A + M_e}{\left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C_{AB}\right]\left(1+\frac{m_A}{L_{AB}}\right) + \left[(C_{AB}+C'_{AB})\frac{m_A}{L_{AB}} + C'_{AB}\right]\frac{m_A}{L_{AB}} + K_r\left[1+\frac{n_A}{144L_{Aa}}\right]\left(1+\frac{n_A}{L_{Aa}}\right)C''_{Aa}}$$

Example

$M_{F'AB} = 156100'^{\#}$ $\quad \frac{m_A}{L_{AB}} = .0363$ $\quad \frac{n_A}{L_{Aa}} = .1475$ $\quad C_{AB} = 9.6$ $\quad K_r = 10$

$M_{F'BA} = -156100$ $\quad \frac{n_A}{144L_{AB}} = .0010$ $\quad C'_{AB} = 7.4$

$V_{oAB}m_A = 30454$ $\quad C''_{Aa} = 7.4$

$$\frac{\theta_A}{K_{AB}} = \frac{156100 + 30454}{(17).0363 + 9.6(1.0363) + [17(.0363) + 7.4].0363 + 10[1.001(1.1475)]\,7.4} = \frac{1946}{K_{AB}}$$

$$M_{AB} = 156100 - [(17).0363 + 9.6]\,1946 = 136215'^{\#}$$

$$M_{BA} = -156100 - [(17).0363 + 7.4]\,1946 = -171700$$

$$M_{Aa} = -10[1.001]\,7.4 \times 1946 = -144150$$

$$\left.\begin{aligned} MJ_{AB} &= (136215)1.0363 - (171700).0363 + 30454 = 165360'^{\#} \\ MJ_{Aa} &= (-144150)1.1475 = -165420 \end{aligned}\right\}\text{Check}$$

$$M_C = 176200 - 153960 = 22240$$

FIG. 3

Attention is directed to Fig. 1. For an accurate solution, the sum of moments should be zero at the center of the Joint J_A instead of at the faces of supports. Also the moments at faces of supports will be correct if computed on basis of clear spans (as used by Professor Maney) modified by use of angle of joint rotation Θ_A obtained by considering equilibrium about center of joint as above and by inclusion of the amount that the end of the members are shifted, normal to their axes, at the faces of support by the joint rotation; i.e., the Δ term. Adhering to the author's terminology, and adding the following:

M_{JAB} = moment at center of joint J_A due to stresses in beam AB at support A and to any loads acting between center of joint and face of support.

M_{JA} = sum of moments from all members and loads at joint J_A as above.

Me = sum of moments about joint J_A due to loads acting between center of joint and face of support A.

V_o = shear from loads on simple beam AB.

Then the equation for moment about center of joint from stresses in member AB and from loads between center of joint and face of support A is:

$$M_{JAB} = M_{AB} + \left[\left(\frac{M_{AB} + M_{BA}}{L_{AB}}\right) + V_o \right] \times M_A + Me$$

$$= M_{AB} \left(\frac{1 + M_A}{L_{AB}}\right) + M_{BA} \frac{M_A}{L_{AB}} + V_o M_A + Me$$

Placing the sum of all moments at a joint center equal to zero, we obtain an equation in terms of Θ_A and known Δs, and moments at faces of supports are obtained by the author's method. The value of Θ_A may differ from that obtained by the author's procedure.

To illustrate the influence of the various factors Fig. 2 and 3 are given, applying the above analysis to the one span and two span symmetrical cases for dead load only as given by the author. It will be noted that for symmetrical loads on the frames under consideration, the moments at all points agree very closely with those obtained by Professor Maney with his clear span assumption, except at top of the end verticals, where the moments given by the writer's assumptions are appreciably larger. This difference illustrates the influence of the height of the vertical and also of the length n. The writer would suggest that the procedure he has used here may be advantageously utilized to estimate the probable error at exterior columns under symmetrical loads, and at other critical points under partial or unsymmetrical loads with the clear span assumption. Finally, it would seem that the center to center assumption would not apply in any case. The error with the center to center assumption is particularly noticeable in the positive moment in the beam spans.

The procedure used by the writer above, may also be employed for other purposes. He has found it advantageous in analyzing secondary stresses in trunnion column frames and the rear end of bascule trusses where the gussets are large compared to the lengths of the members.

Assumption No. 6—Rib Shortening from Shrinkage and Temperature Effects—It seems desirable that more accurate information be obtained by experiment or direct measurement on the amount of rib shortening to be expected from these two sources. In 1915, F. R. McMillan established experimentally that shrinkage of a serious amount takes place in concrete after setting.[5] In 1929-30, Professor Maney made extensive investigations on this subject and not only measured shrinkage separately from plastic flow but determined the quantitative value of the former.[6] Professor Maney's experiments were performed in a laboratory, protected from the weather, and on relatively small sections compared to the rigid frame bridges under consideration here. It would seem to the writer that the shrinkage to be expected in a structure built under weather conditions and of rather massive sections, would not necessarily be nearly so large as that found by the author in his investigation. It is to be hoped that the author will find it possible to extend the investigation to cover such factors as the relative size of section, the effect of weather and of frequent wetting from rain or ground water in fills, etc.

Similarly the effect of temperature should be more definitely known in terms of size of structure, location geographically, and time of year during which construction takes place. The effect of temperature should be measured from the temperature at which permanent set first takes place when the concrete is poured, and this is not necessarily at all the normal temperature of the locality.

In this connection, the writer wishes to commend the author for his very practical and desirable suggestion for allowing adjustment in the structure of three span or more units for these effects.

The author has rendered a service to the engineering profession by presenting in clear and concise form in one paper a general solution of the rigid frame problem by this basic and very useful method.

H. G. Overholt (Asst. Bridge Engineer, Minnesota Highway Dept., St. Paul, by letter): Professor Maney's paper is the most valuable contribution from the designer's standpoint on rigid frame analysis that has come to hand. The treatment of the single span problem with it's complete illustrations and comparisons and the useful constants supplied in the charts of Fig. 1, 2, 5 and 6 covering both uniform and standard truck loadings, permits a rapid determination of a suitable

(5) "Shrinkage and Time Effects in Reinforced Concrete," F. R. McMillan, Bulletin No. 3, Engineering Studies, University of Minnesota, March 1915.

(6) "Shrinkage of Concrete as a Factor in Compressive Steel Stress," G. A. Maney, *Engineering and Contracting*, August 1929, June 1930.

frame for final analysis. Although this paper deals with the slope deflection method, the writer's conclusion is that the two-hinged arch analysis is preferable for final analysis for the single span problem, since no corrections for camber are needed. It is clearly unnecessary to resort to any method of successive approximations when only one unknown (value of horizontal thrust) is sought which is the case for symmetrical single spans. The complete solution given in Fig. 9 confirms the above opinion.

In the twin-span and triple-span cases dealt with in Case 2 and Fig. 12, 13 and 14 and onward, the problem of design takes on a different aspect. The two-hinged arch method becomes laborious and the method of moment distribution, which contains no inherent check, leaves one with the feeling that a little prayer should be appended to the computations before they are filed away. The designer may then turn to the slope-deflection method as illustrated in Mr. Maney's paper. The procedure is direct and relatively simple, but more important than this is the fact that the necessary physical concepts and visualizations may be carried along with the set-up of the equations.

Joseph A. Wise (*Associate Professor of Structural Engineering, University of Minnesota, by letter*): The paper presented by Professor Maney is a fine contribution to the subject and enables the slope-deflection method to be applied with ease in practice.

The curves given in Fig. 1 and 2 are compressed into a small space and are difficult to read with sufficient accuracy. It is suggested that larger diagrams would be helpful. The curves for k_3 and k_5, Fig. 1, are not quite accurate.

The values of the various constants can be calculated from the following equations, which were derived by direct integration of the $\frac{M}{I}$ curves.

In Fig. 1,

$$C_{AB} = \frac{(3r-2)\, r^2R + 3r^2 - 4}{(r^2R + r - 2)\,(3r^2R + 3r + 2)}\, 4r^2$$

$$C'_{AB} = \frac{(3r-4)\, rR + 3r - 2}{(r^2R + r - 2)\,(3r^2R + 3r + 2)}\, 4r^3$$

In these and in subsequent formulas

$$R = \frac{\tan^{-1}\sqrt{r-1}}{\sqrt{r-1}} = 1 - \frac{1}{3}(r-1) + \frac{1}{5}(r-1)^2 - \frac{1}{7}(r-1)^3 \quad . \; . \; .$$

$$k_1 = \frac{r}{48\ (r-1)} \cdot \frac{3r\ (r^2 + 4r - 8)\ R + (3r^2 + 14r - 8)}{3r^2R + 3r + 2}$$

$$k_3 = \frac{r}{8\ (r-1)} \cdot \frac{r\ (3r - 4)\ R + 3r - 2}{3r^2R + 3r + 2}$$

$$k_5 = \frac{r}{4} \cdot \frac{3rR + 1}{3r^2R + 3r + 2}$$

In Fig. 2,

$$C_{AB} = \frac{r^2}{2} \cdot \frac{(2\log_e r + (r-3)\ (r-1)}{(r+1)\ \log_e r - 2(r-1)}$$

$$C'_{AB} = \frac{1}{2r} \cdot \frac{r^2 - 1 - 2r\ \log_e r}{(r+1)\ \log_e r - 2\ (r-1)}$$

$$C''_{AB} = \frac{r^2\ (r-1)^3}{2r^2\ \log_e r - 3r^2 + 4r - 1}$$

$k'_1 =$ [illegible] $r + 2 \log_e r$ [illegible] $\frac{5r-1}{2r\ (r-1)}$ / [illegible] $\log_e r \cdot \frac{3r-1}{r^2}$

$k'_2 = \frac{\text{[illegible]}}{6}$ [illegible] $\log_e r - \frac{11r^2 + 2r - 1}{2r\ (r-1)^2}$ / [illegible] $(r-1)$ [illegible] $\log_e r - \frac{3r-1}{2r^2}$

AUTHOR'S CLOSURE

The detailed analysis of end conditions as affecting "span length" assumptions, is given a thorough and excellent treatment by L. T. Wyly and indicates that in some cases neither a clear span nor a center-to-center span length is sufficiently correct. Of course, there are many cases such as that of wind loading involving "bent" action or translation and rotation of the member as a whole, where "center-to-center" length assumptions are imperative in dealing with the loads.

Mr. Overholt calls attention to the really important function of the paper, which is to simplify the analyses of the multiple span cases which should become much more common among bridges of the immediate future. The author hopes soon to be able to present curves similar to those presented for highway loadings, which will simplify calculations for railroad loadings.

The formulas for constants suggested by Mr. Wise are very interesting and should be helpful in the plotting of data for use in individual

design offices. It should be remembered that C, C' and C'' values are strictly relative, rather than absolute constants, and will permit of considerable variation without producing appreciable differences in critical moment values.

Discussion of a paper by F. R. Watson and Keron C. Morrical:

"Sound Absorbing Value of Portland Cement Concrete"*

CONVENTION DISCUSSION

Walter H. Wheeler (Minneapolis, Minn.): I should like to ask Professor Watson, is it not possible to get too much absorption in a room so that it appears to be deadened?

F. R. Watson: The answer to that is generally no. The reason why it seems too dead is because, particularly with a musician, he is unable to hear himself; he seems to get a ringing effect, as in a bathroom. If you can equip the stage so that the performer can hear himself, he does not care how dead the rest of the room is. On the other hand the listeners find it very important how dead the room is, because experiments show with they can hear better, they can understand speech better the deader it is, and the reason why the complaint has come up about a room being too dead is exactly as I have set forth. If you can equip the stage with reflecting walls that will throw the sound back at the musician so he can hear himself, he is satisfied, but the listeners want to be satisfied in a different way.

Roy L. Peck (Western Brick Co., Chicago, Ill.): Will Professor Watson explain why the coefficient of sound absorption will vary higher and lower than the average with a given material? First slide (Table 1) shows a variation there with the same material.

Professor Watson: Generally speaking, the materials which are thicker will absorb more sound at the low frequencies. With a high pitch, very thin material will absorb sound, and the same material of increased thickness, does not help you any. For low frequencies, it has a marked effect, because if you start with a thin material you will get one coefficient; if you double the thickness of the material, absorption is not doubled but certainly it is greater. These tiles are four inches thick, which is an unusual thickness for acoustical material, and I think it is for that reason it shows so good an absorption at the low frequencies. If it were a thin material, half an inch thick, for

*Journal, Amer. Concrete Inst., May-June 1936; *Proceedings*, Vol. 32, p. 659.

instance, it would not have so large a coefficient. I might say also that the length and diameter of the pores has a certain relation to the frequencies, and some theoretical work has been carried out, but the experimental work and verification are difficult under any circumstances, and we have no reliable results to show exactly what that relation is. The thicker materials however will give more absorption at the low frequencies and will tend to make the frequency across the whole range constant.

Benjamin Wilk (*Manager Standard Products Co., Detroit*): In the last couple of years approximately half a dozen motion picture theaters have been built in Detroit using combination walls. Now a couple of questions as to the relationship between the back wall and the side wall—is the back wall a more effective sound deadener than the side wall? What effect has the shape of the auditorium? What effect has size and depth of openings in wall material?

Professor Watson: Generally speaking the walls remote from a speaker can be treated with absorbing material but not those near the speaker because the speaker likes to hear himself. If you have a wall that is a good reflector, you will get an echo. The auditors near the front will get the sound directly and then w[illegible]y reflection from a distant wall which is a disturbing factor. Curved walls are always dangerous for acoustics; they concentrate the sound. The reflected sound can be stronger than the direct sound because it is concentrated, and I have yet to find a case where curved walls help the acoustics; they are usually a hindrance; you have to apply special means to get rid of their defects. Regarding the size of the openings in the absorbing medium, that is a matter for tests. There is evidently a maximum efficiency to be determined experimentally. We have some theories in connection with it which will help to some extent, but I believe that the ultimate answer to that question would have to be found experimentally. If you take a certain material and vary the size of the holes and their distance apart, you can readily, by a few experiments such as Mr. Morrical has carried out, find out which material is the best and whether or not you have gone too far.

Mr. Wilk: Generally speaking, though, the coarser the texture, the better? Or is a medium texture better than a coarse texture?

Professor Watson: Well, it really must be a combination of the two; you must have large channels to get the sound in and then you must have fine channels on the interior; in that sense, a coarse texture is better than a fine. If you take little sand kernels which are very close together, it would not be as satisfactory as a material which has

large holes in it with the little holes radiating out from these large channels on the interior.

Question: There have been a great many experiments performed on the sensitivity of individuals to sound. I happen to be connected with a few of them at Northwestern University, just as an assistant in the psychology department. What I wanted to know is about the transmission of sound through material, that is from one room to another room. Every big building with machinery seems to have that problem. In the Tribune Tower it was evidently very important, and there is a lot of work being attempted along that line. I was wondering what was the fundamental basis of the work and how would concrete do to prevent sound traveling from one room to another?

Professor Watson: I purposely did not say anything about sound transmission; that is the other fifty per cent of the problem. Generally speaking, to stop sounds from going from one room to another, you ought to have something massive in between; that is pretty nearly the whole answer to it. It is possible, by certain ingenious devices, to use something that vibrates. If you can get its period of vibration in accord with the incidence of sound, you can sometimes set up a thinner material which is efficient, but generally speaking the thing to stop sound between two rooms is an eight inch masonry wall. I think I should add that there is this possibility with the four inch porous concrete tile we have been working with. We made a few experiments in which we filled the core of the material with dense concrete so that the sound on either side would be absorbed by this open, porous structure, but when it hit that center core, which was dense and massive, it was stopped from going through. We found that by using a tile of the usual porous nature, the sound would go right through it; that is one disadvantage with haydite tile, the sound goes right through, but if you fill up the core with a dense concrete, you can decrease the transmitted sound by 25 decibels, and that is a lot. It is absorbent on the two sides but stops the transmission in the middle.

Discussion of a report of Committee 710:

"HIGH EARLY STRENGTH CEMENTS IN CONCRETE MASONRY MANUFACTURE"*

CONVENTION DISCUSSION

Harry C. Shields (Pennsylvania-Dixie Cement Corp., New York City): I should like to ask Mr. Wilk with reference to his July, October and December tests—as to the methods used to control the manufacture of the test specimens, with particular reference to the tamping and the proportioning.

Benjamin Wilk (Chairman, Committee 710): As stated, the methods used in making the tests were the same as referred to in detail in the paper last year.† We have batch mixers; cement was weighed; the tamping was usual manufacturing practice; the same operator was used in making all the different specimens.

Mr. Shields: I think too much significance should not be attached to the results of the 1935 tests, in view of the apparent lack of control with reference to factors that would very readily influence results. It has been my experience in years gone by in the manufacture of concrete products, that the number of tamping blows delivered on a concrete block very materially influenced its strength and some other properties. It is observed that the tamping blows on these block, the strength results of which you attempted to compare, varied from 9 to 12 on each block; also that the aggregates were measured by eye by permitting them to run down a chute, and mixing water was also a matter of guess work. I believe that such slipshod methods could not possibly provide the necessary degree of accuracy and control that would justify the conclusions drawn in your report.

Mr. Wilk: The test results are all based on a yield of 20 blocks per sack of cement. Our variation was less than one block. All results have been referred back to a 20-block per sack base, so that even in a

*JOURNAL, Amer. Concrete Inst., May-June 1936; *Proceedings*, Vol. 32, p. 673.
†"High Early Strength Cements in Concrete Products Manufacture," by Benjamin Wilk, JOURNAL, Amer. Concrete Inst., Jan-Feb. 1935; *Proceedings*, Vol. 31, p. 241.

batch from which more blocks or less blocks were made, the strength variation would be practically a straight line; by putting it back to a 20-block yield, we have an effective comparison.

It does not take long for an experienced operator with a one-sack batch mixer (don't forget that, those familiar with the Blystone mixer) to get practically the same number of block per batch; we would be surprised if our mixer man could not do that; all of our results last year were very close.

George P. Dieckmann (Mason City, Ia.): With 70 pounds of high early strength cement or 94 pounds of standard cement—what are the effects on plasticity and workability?

Mr. Wilk: The same in both cases; we put in enough water to give us the same consistency; a water web mark on the outside, the amount of water actually used is practically the same in the batch, even though you use less high early strength cement.

J. H. Chubb (Penn-Dixie Cement Corp., New York City): We made some interesting tests this year comparing standard with high early strength cement in making cinder block. At the end of a day's run, using high early strength cement, we had 2,400 blocks. Using six and a half sacks of the standard cement against five sacks of high early strength, to a batch at the end of a day's run we had 2,490 blocks; 6½ sacks of standard cement gave us 83 blocks; 5 sacks of high early, 80 blocks. Total labor and material cost for the standard block as compared with the high early strength block was six dollars in favor of the high early strength cement. But we had 90 more standard blocks that sold at the yard for 15½ cents each, so that we had a gain in selling price of $13.95. Subtracting additional cost for labor and materials ($6.00), and there remains an advantage of $7.95 in favor of standard cement. We tested five blocks of each kind at 28 days. The high early strength block just failed to meet a 1,000 p.s.i. requirement; the standard cement block averaged 1,100 p.s.i., testing a little over 11 per cent more than the high early strength block. Thus with existing prices for material and labor, we showed a saving of $7.95 per day on a block that tested 10 per cent above requirements as compared with one that failed to meet specifications with the high early strength cement.

Mr. Wilk: Joe Chubb and I are good friends. He mentioned this to me yesterday so that I could think about it, and I appreciate that, because offhand I would not be able to analyze it the way I am going to analyze it right now. He said it cost him $6.00 more to use standard portland cement than high early strength cement, but he got 90 blocks more—90 blocks for an additional $6.00—or about 6½ cents each.

Mr. Chubb said the selling price was 15½ cents. As a practical products manufacturer, I would not compare the cost of block with my selling price. My selling price includes many other items not included in this comparison. The cost of a block to me is raw materials and labor as the finished block go into the yard. My overhead is the same whether I use standard portland cement or high early strength cement; the sales cost is the same; insurance is the same. The only comparison I can make legitimately is of actual costs of direct labor involved plus cost of the cement and aggregates. The cost of a unit is about three cents for cement and about two cents for aggregate—a total of five cents. Three more blocks per batch would cost about 15 cents, but the additional cement cost was 20 cents. Is that in favor of standard portland cement?

Chairman Pearson: Among my friends in this group here there are several who enjoy the solution of trick problems. I have a weakness for them. I think this problem put up by Mr. Chubb is a trick problem. The question I would ask is this, if Mr. Chubb is going to put in the selling price of those three extra blocks, why not pay a nickel and make it up? That is, in the raw material. If you are getting a yield that is less, that means the volume of your batch is actually less than in other cases, and I believe five cents' worth of aggregate would probably make up the three blocks.

Mr. Wilk: Mr. Chubb's tests were at 28 days. That may be all right from the standpoint of the man selling cement, but a products manufacturer doing a fair amount of business, doesn't give blocks 28-day storage. I don't expect any more strength at 28 days with high early strength cement than with standard cement, the advantage is in earlier results. Manufacturers know what it means to have a unit strong as it comes out of the curing room; it means saving in culls, easier handling and shorter storage time. What about sending blocks out to a job at 7 or 10 days? Last month in the city of Detroit, the average block delivered on the job was not 10 days old. Suppose a plant making 5,000 units a day has to store them for 28 days—consider the space required, the distance to be moved with so large a storage area to the outer edges of the storage yard.

Mr. Shields: Irrespective of the advantages that might be gained in handling block from the steam room, do you, in the light of your experience as a products manufacturer, deem it sound practice to deliver block to the job after only a 7 or 8-day curing period? In pursuing such practice one should carefully weigh the detrimental effect of subsequent shrinkage when these units are placed in the wall.

Mr. Wilk: Strength indicates what has happened. Twenty-eight days is no magic period—28-day curing in December or in January is different from 28 days in August. It depends on how you cured your units and how they are stored afterwards. We are not suggesting a panacea for the products manufacturer's ills. We are indicating what can be done with different cements, high early strength cement as against standard cement. His curing conditions should be such that he gets a satisfactorily cured unit with a moisture content when delivered on the job that will meet requirements in this respect.

Earl A. Soloman (Penn-Dixie Cement Corp., New York City): Have you any reason to believe that block made with high early strength cement will have any less moisture at six, seven, eight or nine days than with standard cement?

Mr. Wilk: We have been listening today to statements that leaner mixtures do not shrink as much as richer ones. I take it the moisture content would not be very much different.

Mr. Soloman: I have some reports covering three plants in New Jersey and one in New York where, in the two or three day range, the regular cement catches up with the high early strength cement and after that it passes it; the units had more moisture at seven days with high early strength cement than with regular cement.

Chairman Pearson: I should like to emphasize that the purpose of the committee is only to put the results of certain tests and plant experience at the disposal of products manufacturers. There is no attempt to do anything except to find the facts. Some of these are not so easy to find, and one of them is this question of yield. You realize that in plant operation when you weigh up a lot of block in succession, there is considerable variation, a variation probably comparable with the difference in the yield between the two types of cement. I do not like to have Mr. Chubb leave the impression that you are going to get four per cent less yield when you use a reduced amount of high early strength cement than when you use regular standard cement. It may have been so in that case, but I would not consider it established unless many careful repeat tests had been run.

We cannot determine such things except from a very large number of cases, but in general we find from our laboratory and field studies that a block made with a reduced amount of high early strength cement, as in these tests, weighs about a pound less than a similar block made with the full amount of normal cement. If this figure is assumed to be approximately correct, then the loss of 1 lb. per block in a 20-block batch about offsets the 24-lb. lower weight of the total

batch, and the yield must therefore be very nearly the same in the two cases.

COMMITTEE CLOSURE BY MR. WILK

The freezing and thawing tests referred to in the report have been completed. Compressive tests were made after 100 cycles of freezing and thawing. They indicated that there is no loss in effectiveness of the 70-lb. high early strength concrete as compared with the 94-lb. normal cement concrete. Tests on the volume change of the two kinds of concrete indicate that there is no marked difference so that it would be immaterial what concrete was used.

The aggregate, however, had a marked effect. The cinder concrete showing greater and the slag concrete considerably less shrinkage than the gravel concrete.

These tests made specially for Committee 710 apparently corroborate the results obtained by Timms on tests made several years ago.

TABLE 1

Type of Cement	Type of Aggregate								
	Cinders			Slag			Gravel		
	Days of Moist Curing Prior to Freezing Test								
	3	7	28	3	7	28	3	7	28
N1	2230	2450	2630	4200	4470	4210	4000	4290	3920
N2	*60†	*110†	2300	4040	4950	4380	4070	4550	4350
H1	1830	1700	2000	4110	4720	5480	4180	4300	4470
H2	*55†	2120	2310	4200	4670	3770	3000	3950	4390

*Lost more than ten per cent in weight and classified as disintegrated.
†Number of cycles.

In a paper by Timms on "Effect of Duration of Moist Curing on the Principal Properties of Concrete,"* in an analysis of freezing and thawing, Mr. Timms develops the fact that a lean mix of high early strength concrete (9 gal. per sack), starting with 1750 p.s.i. concrete, will withstand 210 freezing and thawing cycles based on loss of weight as compared with 90 cycles for a rich mix of normal cement (6 gal. per sack), which also starts with 1750 p.s.i. concrete.

Three thousand five hundred-fifty p.s.i. concrete (a lean high early strength, 9 gal concrete), gives 220 cycles as compared with 230 cycles for a 3800 p.s.i. concrete (a rich mix of normal portland cement, 6 gal. concrete).

This would indicate that there would be no disadvantage in using the leaner high early strength cement concrete as compared with the

*Proceedings A. S. T. M., Vol. 34, Part 11, p. 329.

richer normal concrete from the standpoint of freezing and thawing.

Timms also compares the expansion at three months as follows:

With normal cement and a rich mix, (5.70 gal.) the expansion is .012 per cent as compared with a high early strength cement lean mix, (8.5 gal.) having an expansion of .004 per cent.

Total shrinkage at the end of an 18 month period for seven-day and three-month moist cured specimens show:

	7 Days	3 Months
Normal Cement	.112%	.092%
High early Cement	.112%	.094%

In an analysis of capillary flow, Mr. Timms points out in this paper, (Fig. 5, page 340) that a concrete made with 9 gal. per sack, using high early strength cement, will be better than a richer concrete with 5.6 gal per sack, using normal cement.

Quoting from this paper, "The data show the effect of more rapid hardening in the high early strength cements on the early development of a given degree of water-tightness."

Discussion of a paper by J. R. Shank:

"The Mechanics of Plastic Flow of Concrete"*

ERRATA

Unfortunate "make-up" blunders were committed in publishing discussion by H. J. Gilkey and George C. Ernst in the May-June Journal. Fig. 11 (page 707) should have carried this caption: "Plain concrete under sustained compressive loading—Gilkey-Vogt tests, U. S. Bureau of Reclamation, at University of Colorado, 1928-29 (specimen of Table 5)." Fig. 13 of this discussion appears at the top of page 705 without any designation. It should have carried this caption: "Plain concrete under sustained flexural loading—Gilkey-Vogt tests, 1928-29 (U. S. Bureau of Reclamation)."—Editor

AUTHOR'S CLOSURE

J. R. Shank: The discussion by Professor Fridenson pertains more to the work of Dr. W. H. Glanville[15] than it does to this paper and it would therefore be more appropriate for him to answer it, but since the writer has studied and checked parts of Dr. Glanville's work he will attempt to show how the difficulties indicated by Professor Fridenson are obviated. Bibliography reference 30 gives a discussion of these fundamentals in considerable detail which was omitted in this paper because of lack of space. This will not be repeated here but an attempt will be made to clear up the difficulties of Professor Fridenson and others by using Dr. Glanville's original paper[15] as a background.

The first statement is an equation representing the conditions during a time increment in the plastic flow life of a reinforced concrete column under a constant or sustained load.

$$f_c \Delta c = \frac{\Delta f_s}{E_s} - \frac{\Delta f_c}{E_c} \text{ where}$$

*Journal, Amer. Concrete Inst., Nov.-Dec. 1935; *Proceedings*, Vol. 32, p. 149. Discussion May-June 1936; Vol. 32, p. 704.

f_c is the unit stress in the concrete at the time of this time increment.

$\Delta\, c$ is the plastic flow change for a *unit stress of unity.*

$f_c\, \Delta\, c$ is therefore the plastic flow change in the column at this time when the unit stress is f_c.

$\Delta\, f_s$ is the unit stress increment in the steel at this time.

$\Delta\, f_c$ is the unit stress increment in the concrete at this time.

E_c is the initial or instantaneous or elastic modulus of elasticity of this concrete. It is *unaffected by the plastic flow* and is assumed to be the same throughout the plastic flow life of the column. The possibility of improvement with age is ignored. It is *not a variable* and comes in only on elastic calculations.

The second term of the right hand side may be thought of as the elastic recovery which follows any reduction of the stress in the concrete of the column. To explain this equation it should only be necessary to state that the plastic flow increment $f_c\, \Delta\, c$ in this reinforced column must equal the algebraic sum of the strain increment in the steel and in the concrete, but the reader is likely to be confused by seeing two concrete deformations $f_c\, \Delta\, c$ and $\frac{\Delta\, f_c}{E_c}$. There may be a tendency to assume that $f_c\, \Delta\, c$ should simply equal the strain increment in the steel $\frac{\Delta\, f_s}{E_s}$ which would only be true if the unit stress on the concrete remained constant, which it does not. If this last assumption were carried to the extreme condition of nearly all of the load being taken by the steel we would have the absurdity of the concrete all but ceasing to be stressed while keeping its full initial strain. It appears that this first statement by Dr. Glanville is correct and complete. It contains three variables c, f_c, and f_s, and no variable modulus of elasticity is necessary.

It is possible to eliminate the variable f_s from the equation so that the change in the total stress in the concrete must equal at all times that of the steel since the external load is constant,

$$\Delta\, f_s A_s = -\,\Delta\, f_c A_c$$

$$\text{Then} \quad f_c\, \Delta\, c = -\frac{\Delta\, f_c A_c}{A_s E_s} - \frac{\Delta\, f_c}{E_c}$$

$$\text{or} \quad \Delta\, f_c = -\frac{f_c\, \Delta\, c}{\frac{A_c}{A_s E_s} + \frac{1}{E_c}} \text{ or to simplify, } f_c = -\frac{f_c\, \Delta\, c}{b}$$

$$\text{then} \quad \frac{df_c}{f_c} = -\frac{dc}{b}$$

Integrating:—$log_e f_c + C = -\frac{c}{b}$

When c the plastic flow $= O$, f_c becomes f_o, the initial elastic stress.

$$C = -\ log_e f_o$$

$$log_e f_c = log_e f_o = -\frac{c}{b}$$

and $f_c = \dfrac{f_o}{ec/b}$ or $f_c = \dfrac{f_o}{e\left[\dfrac{A_s}{A_s E_s} + \dfrac{1}{E_c}\right]}$

It appears that Doctor Glanville very skillfully side-stepped the variable modulus of elasticity and has developed a simple and accurate expression and that Professor Fridenson's criticism does not hold.

The last sentence of Professor Fridenson's discussion indicates that he has lost sight of the fact that the plastic flow y in the formula given by the writer as well as c of Dr. Glanville's work represent the plastic flow for a unit stress of unity and that it is only necessary to multiply by any unit stress, constant or variable, to represent a plastic flow for any stress.

The writer is grateful to Professor Gilkey and Mr. Ernst for the fine contribution which they submitted as a discussion. It is to be hoped that they, as well as others will continue this work as there is much still to be done.

ERRATA

Bibliography No. 3, A. *H*. Fuller instead of A. *N*. Fuller.
Bibliography No. 16, *Trans.* not *Proc.*
Bibliography No. 25, Andersen not Anderson.

Discussion of a paper by F. E. Richart:

"A Study of the Economics of High Strength Concrete in Building Construction"*

George Robert Wernisch (Concrete Reinforcing Steel Institute Research Fellow, Lehigh Univ., Bethlehem, Pa., by letter): The writer was particularly interested in that portion of Professor Richart's paper pertaining to the use of higher steel stresses. Professor Richart mentions the objections to the use of higher steel stresses: larger deflections, increase in tensile cracks and a consequent weakening in bond and shear resistance. Test data of 33 slabs tested at Fritz Laboratory, Lehigh University, under immediate charge of the writer, indicate that deflections and cracking are not excessive at steel stresses of 30,000 p. s. i. The bond resistance is not seriously reduced by the small, hairline cracks which result at a steel stress of 30,000 p. s. i.

Referring to the author's Table 3—Cost Data for One-way Slabs—the writer (assuming a thickness of slab equal to the effective depth plus 1.25 in.) found that by designing the reinforcement for 30,000 p. s. i. the total cost of the slab (effective depth 7.50, 2,000 p. s. i. concrete) was approximately 8½ per cent lower than when using a 20,000 p. s. i. steel working stress.

The cost difference is more pronounced in the smaller depths (effective depth of 4.1); when using 6,000 p. s. i. concrete and 30,000 p. s. i. steel working stress, the total cost of the slab is 20 per cent less than when using 6,000 p. s. i. concrete and 20,000 p. s. i. steel working stress.

Before investigating the feasibility of high strength concrete it would appear more reasonable to investigate the potential strength of present day concrete. The common belief seems to be that in design, steel, with a factor of safety of approximately 2, is balanced with concrete having a factor of safety of 2.5. Actually this is not the case. Slater and Lyse found that the ultimate concrete compressive strength in flexure, when assuming a straight line variation in stress, was about 1.5 times the compressive strength of the concrete cylinders. To the writer it appears to be illogical to balance steel with a factor of safety

*Journal, Amer. Concrete Inst., Mar.-Apr. 1936; *Proceedings*, Vol. 32 p. 459.

of 2 against a concrete factor of safety of approximately 3.75 in slabs and beams. The science of concrete making has progressed sufficiently to warrant a factor of safety of 2.5 for concrete, based on actual ultimate flexural compressive strength, rather than on cylinder strength. If one desires to use a concrete working stress of 1,200 p. s. i. it is necessary to use 3,000 p. s. i. concrete under present code regulations, whereas data on this matter indicate that 2,000 p. s. i. concrete should be used with a factor of safety of 2.5, thus obtaining a saving of about 4 per cent in the concrete cost, and reducing volume changes, which may be important in high strength concretes.

In the calculations given herewith it was assumed that a 60,000 p. s. i. yield point steel would cost $8 per ton more than the figures used by Professor Richart. The extra cost probably would be even less.

CONVENTION DISCUSSION

W. M. Dunagan (*Iowa State College, Ames, Iowa*): I wish to supplement Professor Richart's paper with some data shown in three figures (Fig. 1-3—presented by stereopticon on the floor of the convention). The work summarized in these figures was done after reading a paper by Professor Lyse[1] and my close contact with the work being done by Professor Gilkey and Mr. Ernst.[2] On noting the title of Professor Richart's paper in the program for this meeting I decided that these data had such a fundamental bearing upon his subject matter that their presentation was justified.

My work referred to and condensed to the three figures was a brief investigation in the order: (1) A careful laboratory design of a series of concrete mixes, (2) physical tests of the concrete prepared from these mixes with results in the form of actual strength and modulus of elasticity, (3) a computation of the materials cost for unit volumes of each concrete, (4) the design of a series of reinforced beams in which the size of the beam was permitted to vary with the quality of the concrete and (5) a plotting of the economic relationships found to exist in the resultant beams.

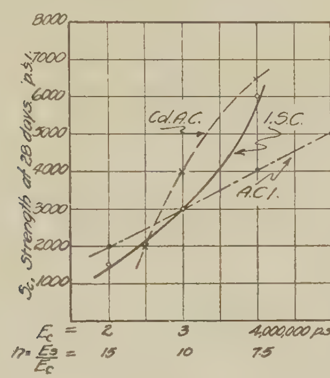
In Fig. 1 the I. S. C. curve presents the strength-stiffness relationships found in this series; this curve is contrasted with two other groups of available data so that agreement may be noted. Fig. 2 is a summary of the mix design data from which the costs of the concrete were estimated. Fig. 3 presents the relative cost of beam designs resulting from the use of these carefully assembled data.

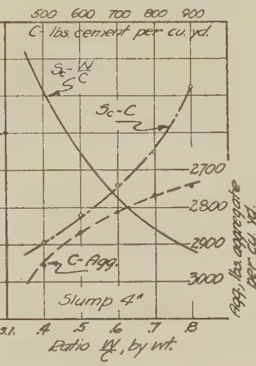
(1) Relation between Quality and Economy of Concrete. Inge Lyse, JOURNAL, Amer. Concrete Inst., March-April 1933, Vol. 4, p. 344.

(2) Report of Project Committee on the use of High Elastic Limit Steel as Reinforcement for Concrete. Gilkey and Ernst. *Proceedings*, Highway Research Board, Vol. 14, Part I, 1935, p. 283-302.

FIG. 1—STRENGTH-ELASTICITY RELATIONSHIPS

FIG. 2—STRENGTH-MATERIAL QUANTITY RELATIONSHIPS





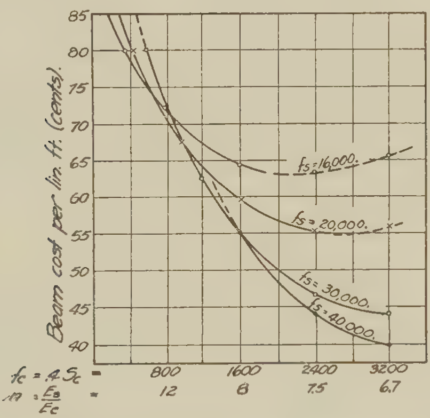


FIG. 3—BEAM COST-WORKING STRESS RELATIONSHIPS. (SUPPORTING STRENGTH AND b/d RATIO CONSTANT)

This is intended to be an example of what might occur in the economics of structural design if concrete mixes are designed by best practice, placed under ideal control conditions and the designer's procedure takes full advantage of this situation. The working stress values for concrete are taken as 40 per cent of 28-day test strengths, the n values are from actual tests and the cost data are for materials cost only. The dotted portions of Fig. 3 are those areas where impractical steel percentages exist. Limiting deflection conditions have not been investigated as a part of these data.

The chief indication of Fig. 3 is that until the design stresses of reinforcing steel are increased as those of the concrete are increased we can not realize the economy possible in taking full advantage of the

potential quality of the concrete. I think the curves are self explanatory; those persons interested in the design of concrete mixes coupled with structural design may discover further interesting indications.

M. O. Withey (Professor of Mechanics, Univ. of Wisc., Madison): In the slide just shown (Fig. 4) I wish to call attention to one feature which I do not think the speaker emphasized, namely that concrete cracks in tension when the unit deformation is approximately 0.0002. When a stress of 30,000 p. s. i. is imposed upon the steel, the corresponding unit strain in the steel and in the concrete adjoining that steel will be 0.001, or five times the strain which will crack the concrete. Professor Richart, in his discussion, pointed out that the use of these high strength mixes and these high strains in steel attending the use of those mixes, would be attended by cracking. I wish to emphasize that in reference to Professor Dunagan's chart the cracking accompanying high steel stresses should be kept in mind.

E. B. Rayburn (Indianapolis, Ind.): Getting back to the economic view and probably from a partisan standpoint, the ready mixed concrete producer is glad to see a higher strength concrete used. He will have less competition from the irresponsible operator and more business.

AUTHOR'S CLOSURE

F. E. Richart. The foregoing discussions emphasize some of the salient features involved with high strength concrete. I agree with Mr. Wernisch that the current working stresses for concrete are conservative; also that the greatest saving in cost of reinforced concrete would result from the simultaneous use of higher stresses in both concrete and steel, but I am still looking for proof that, in general, deflections and cracking would not be excessive if a steel stress of 30,000 p. s. i. were used. Professor Withey apparently agrees with this viewpoint. While I have not had the chance to study Professor Dunagan's chart, the conclusions from it seem to be in harmony with what has been said regarding possible economies.

The desire of ready-mixed concrete producers to sell high quality concrete is commendable; they have an excellent opportunity to produce and deliver to the job a factory-made and controlled material of high quality and uniformity.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 32—1936

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 7, Sept.-Oct., 1935 to May-June 1936

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the main classifications as follows:

ARCHITECTURAL DESIGN

BIBLIOGRAPHIES

BREAKWATERS—DOCKS AND SEA WALLS

BRIDGES

BUILDING CODES

BUILDINGS

BUILDING UNITS

CENTRAL MIXING

COMMITTEES

CONSTRUCTION—DETAILS, METHODS AND COSTS

CURING

DAMS

DURABILITY

ENGINEERING DESIGN

FAILURES

FIRES, FIRE TESTS AND FIRE-PROOFING

LAWS AND PROPERTIES

MASONRY UNITS

MATERIALS AND TESTS

MIXTURES

PAVEMENT

PERMEABILITY

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PRECAST UNITS

RAILWAY USES

RESEARCH

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